Thos. Telford

Weale, J.

THE

THEORY, PRACTICE, AND ARCHITECTURE

OF

BRIDGES

OF STONE, IRON, TIMBER, AND WIRE;

WITH

EXAMPLES ON THE PRINCIPLE OF SUSPENSION:

ILLUSTRATED BY

One Hundred and Thirty-eight Engravings

AND NINETY-TWO WOODCUTS.

VOLS. I. II.

RE-ISSUE, WITH SOME EMENDATIONS,

FACED BY A PORTRAIT OF

THOMAS TELFORD, C.E.

CONTENTS OF THESE VOLUMES—TEXT.

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1855-56.

ADVERTISEMENT.

This work, commenced in 1839, and continued at intervals which have probably exhausted the patience of many of the Subscribers, is at length completed,—the object at first contemplated having been attained by a concentration of talent on this important branch of science, and by the production, it is hoped, of a work on the Art of Bridge Building which will supply the vacant space in the library of the Practical Engineer.

Since the time of Hutton, Attwood, &c., the Theory of the Arch has been deeply investigated in this and other countries; and as the principles of Coulomb and others have caused such investigations to progress extensively, the production of a work on this important subject, combining numerous Theoretical and Practical Examples down to the present period of improvement in the art of construction, necessarily required no inconsiderable portion of time, labour, and expense.

The Practical Treatise on Bridge Building will, it is anticipated, together with the Specifications and the Paper on Foundations, supply all that is desirable for the Student in the pursuit of his knowledge of the Art; while the great variety of engraved specimens by which the work is illustrated, and the

numerous exemplifications of comparative construction, together with the lucid and practical description of them which is appended, will prove interesting to the Engineering public, —and thus compensate for the unavoidable delay which has occurred.

The Analytical List of Contents will guide the reader to the several divisions of the work; and the General Index which I have supplied will be found useful for all the objects of reference.

JOHN WEALE.

59, High Holborn,
Feb. 14, 1843.

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PAGE	LINE	ERROR	CORRECTION
48	11 from bottom	to exceed	to be less than
„	6 „	as low	as high
„	5 „	as high	as low
108	10 from top	on account of the amount	on account of the smaller amount
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LIST OF PLATES.

Making One Hundred and Thirty-eight Plates,
With Ninety-two Wood-cuts.

THEORY OF BRIDGES.

This very important subject has exercised the talents and ingenuity of some of the greatest mathematicians in modern times, and many different solutions have been given to the various problems connected with it; but, as the greater part of them are founded on suppositions that have no existence whatever either in nature or practice, they have had a tendency rather to mislead than direct those who are engaged in the operations of Bridge Building.

Since the time when Lord Bacon overthrew the absurdities of Aristotle, and showed to the world that experience was the only true guide to philosophy, it might have been expected that theory and practice would have gone hand in hand,—but this unfortunately has not been the case; for we find that their respective advocates have been continually cavilling with each other.

It is to be regretted, deeply regretted, that theoretical and practical men should have always been thus opposed, and have looked upon each other's efforts rather with contempt than admiration, though they seem evidently to have been designed for each other's mutual aid;

and nothing but a deep-rooted prejudice could have continued a system of opposition so destructive of the best interests of society. Our illustrious countryman, Dr. Olinthus Gregory, in the preface to his excellent work on Mechanics, states with his usual elegance, that "theoretical and practical men will most effectually promote their mutual interests, not by affecting to despise each other, but by blending their efforts; and further, that an essential service will be done to mechanical science, by endeavouring to make all the scattered rays of light they have separately thrown upon this region of human knowledge converge to one point."

That a theory may be properly tested, too many facts cannot be collected, too many energies cannot be exerted; for however beautiful may be the theory as far as abstract science is concerned, and however legitimately may the consequences flow from the premises, yet if these premises are not in strict accordance with what is known to take place in actual practice, such theory must ultimately be abandoned, and give place to that which is so founded as to agree with the results of experience and observation. Gauthey, speaking of the theory of La Hire, observes that such analytical researches are unfortunately founded on hypotheses which every day's experience contradicts.

We shall in the first place give a brief account of the principal writers on the equilibrium of the arch, with some notice of their theories.

In 1691, the celebrated mathematicians, Leibnitz, Huygens, James and John Bernoulli, solved the problem

of the catenary curve: it was soon preceived that this was precisely the curve that should be given to an arch, the materials of which were infinitely small and of equal weight, in order that all its parts may be in equilibrium. In the Philosophical Transactions for the year 1697, it appears that David Gregory first noticed this identity, but his mode of argument, though sufficiently rigorous, appears not to be so perspicuous as could be desired.

In one of the posthumous works of James Bernoulli, two direct solutions of this problem are given, founded on the different modes of viewing the action of the voussoirs: the first is clear, simple, and precise, and easily leads to the equation of the curve, which he shows to be the catenary inverted; the second requires a little correction, which Cramer, the editor of his works, has pointed out.

In 1695, La Hire,[1] in his Treatise on Mechanics, laid down from the theory of the wedge, the proportion according to which the absolute weight of the materials of masonry ought to be increased from the key-stone to the springing in a semicircular arch. The historian of the Academy of Sciences relates in the volume for the year 1704, that Parent determined on the same principle, but only by points, the figure of the extrados of an arch, the intrados being a semicircle, and found the force or thrust of a similar arch against the piers.

In the memoirs of the Academy of Sciences for the year 1712, La Hire gave an investigation of the thrusts

[1] Mr. Attwood has written a dissertation on the construction of arches on the same principles as La Hire.

in arches under a point of view suggested by his own experiments: he supposed that the arches, the piers of which had not solidity enough to resist the thrust, split towards the haunches at an elevation of about 45 degrees above the springings or impost; he consequently regarded the upper part of the arch as a wedge that tends to separate or overturn the abutments, and determined, on the theory of the wedge and the lever, the dimensions which they ought to have to resist this single effort.

Couplet, in a memoir composed of two parts, the first of which was printed in the volume of the Academy for 1729, treats of the thrusts of arches and the thickness of the voussoirs, by considering the materials infinitely small, and capable of sliding over each other without any pressure or friction. But, as this hypothesis is not exactly conformable to experiment, the second part of the memoir, printed in the volume for 1730, resumes the question by supposing that the materials have not the power of sliding over each other, but that they can raise themselves and separate by minute rotatory motions. It cannot however be said that Couplet has added materially to the theories of La Hire and Parent, and none of them treated the subject either in theory or practice in such a satisfactory manner as was afterwards done by Coulomb.

In a subsequent volume there is a memoir by Bouguer on the curve lines that are most proper for the formation of the arches of domes. He considers that there may be an infinite number of curve lines employed for this purpose, and points out the mode

of selecting them. He lays it down uniformly that the voussoirs have their surfaces infinitely smooth, and establishes, on this hypothesis, the conditions of equilibrium in each horizontal course of the dome, but has not given any method of investigating the thrusts of arches of this kind, nor of the forces that act upon the mason-work when the generating curve is subjected to given conditions.

In 1770, Bossut gave investigations of arches of the different kinds, in two memoirs, which were printed among those of the Academy of Sciences for the years 1774 and 1776: he appears to have been engaged in this in consequence of some disputes concerning the dome of the church of St. Genevieve (recently the French Pantheon), begun by the celebrated architect Soufflot, and finished from his designs.

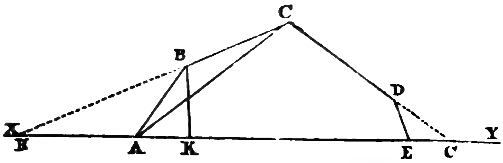
In 1772, Dr. Hutton, late Professor of Mathematics in the Royal Military Academy, Woolwich, published his principles of Bridges, in which he investigated the form of curves for the intrados of an arch, the extrados being given, and *vice versa*. He set out by developing the properties of the equilibrated polygon, which is extremely useful in the equilibrium of structures.

ON THE EQUILIBRATED POLYGON.

Any number n of bars or beams A B, B C, C D, &c., of given weights ω_1, ω_2, ω_3, &c. . . . . ω_n, and supporting given weights W_1, W_2, W_3, &c., at the respective points B, C, D, &c., are freely moveable about

each other in the same vertical plane A B C D, &c., by means of joints at those points, and are supported at the extremities of the first and last, and acted on by gravity. It is required to investigate expressions for determining their respective positions, when in a state of equilibrium.

Considering the two bars AB and BC as fixed at their extremities A and C, their positions are determined as they can form but one triangle ABC upon the constant side A C. But



supposing the three bars A B, B C, and C D, as supported at A and D, their positions become indeterminate, that is, they are moveable about those points, and an equation is necessary to find their respective positions when in a state of equilibrium. We shall, in the first place, take the three bars A B, B C, and C D.

Produce C B, C D, &c., to meet the horizontal line X A Y at B′, C′, &c., draw B K perpendicular to this line, and let B A Y, the elevation of A B to the horizon $=\theta_1$, B B′ Y; that of B C $=\theta_2$; C C′ Y, the elevation of C D $=\theta_3$, &c.

Suppose the weight W_1 to be fixed to the end of AB, and let the vertical pressure acting at B upon A B in direction B K, and upon B C in direction K B $=p$, the lateral force acting at B upon A B in direction K A, and upon B C in direction A K, being equal to that acting at C upon B C in direction K A $=f$.

Then the forces p and f united with gravity having no effect to turn AB about A, we have the equation

$$\left(\frac{\omega_1}{2}+W_1\right)\cos.\theta_1+p\cos.\theta_1-f\sin.\theta_1=0,$$

$$\therefore \frac{\omega_1}{2}+W_1+p-f\tan.\theta_1=0.$$

And since the tendency of the same forces at B, combined with gravity, to turn BC about the joint C, is destroyed in the position of equilibrium, we have also

$$\frac{\omega_2}{2}\cos.\theta_2-p\cos.\theta_2+f\sin.\theta_2=0,$$

$$\therefore \frac{\omega_2}{2}-p+f\tan.\theta_2=0.$$

To this equation add the former, and we have

$$\frac{\omega_1+\omega_2}{2}+W_1-f(\tan.\theta_1-\tan.\theta_2)=0,$$

$$\therefore f=\frac{\frac{1}{2}(\omega_1+\omega_2)+W_1}{\tan.\theta_1-\tan.\theta_2}.$$

Now since the centre of gravity of BC is at rest, the lateral forces at B and C in opposite directions must be equal

$$\therefore f=\frac{\frac{1}{2}(\omega_1+\omega_2)+W_1}{\tan.\theta_1-\tan.\theta_2}=\frac{\frac{1}{2}(\omega_2+\omega_3)+W_2}{\tan.\theta_2-\tan.\theta_3}\dots$$

$$=\frac{\frac{1}{2}(\omega_{n-1}+\omega_n)+W_{n-1}}{\tan.\theta_{n-1}-\tan.\theta_n}\quad\dots\dots\dots\quad(F)$$

consequently

$$\tan.\theta_1-\tan.\theta_2=\frac{1}{f}\left(\frac{\omega_1+\omega_2}{2}+W_1\right)\quad\dots\quad(1)$$

$$\tan.\theta_2-\tan.\theta_3=\frac{1}{f}\left(\frac{\omega_2+\omega_3}{2}+W_2\right)\quad\dots\quad(2)$$

$$\tan.\theta_3 - \tan.\theta_4 = \frac{1}{f}\left(\frac{\omega_3+\omega_4}{2} + W_3\right) \quad . \quad . \quad . \quad (3)$$

&c. &c. &c. &c.

$$\text{and } \tan.\theta_{n-1} - \tan.\theta_n = \frac{1}{f}\left(\frac{\omega_{n-1}+\omega_n}{2} + W_{n-1}\right) \ldots (n-1)$$

By means of these $n-1$ equations involving $n+1$ unknown quantities, *viz.*, $\tan.\theta_1$, $\tan.\theta_2$ . . . $\tan.\theta_n$ and f, the positions of any number of bars may be obtained so as to render them in a state of equilibrium, having previously given two more data, *viz.*, the positions of any two of them: thus suppose θ_p and θ_q the elevations of the p^{th} and q^{th} bars to be given, then if we begin at equation (p), and add all the equations up to the $(q-1)^{th}$ we have

$$\tan.\theta_p - \tan.\theta_q = \frac{1}{f}\left\{(\omega_p+\omega_{p+1} \ldots \omega_q) - \frac{\omega_p+\omega_q}{2} + (W_p+W_{p+1} \ldots + W_{q-1})\right\},$$

$$\therefore f = \frac{(\omega_p+\omega_{p+1}..\omega_q) - \frac{1}{2}(\omega_p+\omega_q) + (W_p+W_{p+1}..W_{q-1})^*}{\tan.\theta_p - \tan.\theta_q}.$$

If in equation (F) we suppose the weights of the bars $=0$, we have

$$\frac{W_1}{\tan.\theta_1 - \tan.\theta_2} = \frac{W_2}{\tan.\theta_2 - \tan.\theta_3},$$

$$\therefore W_1 : W_2 :: \tan.\theta_1 - \tan.\theta_2 : \tan.\theta_2 - \tan.\theta_3.$$

Hence the weights W_1, W_2, &c., are as the difference of the tangents of the angles which the bars make with the horizon, the same as the proportion of the

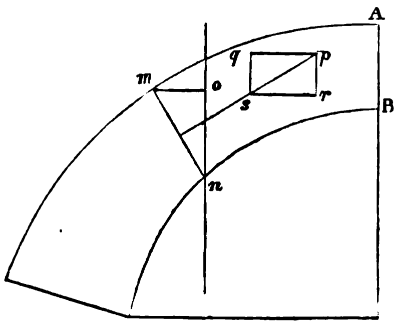
* For the equilibrium of any number of bars without weights, an investigation similar to the above has been given by that able mathematician, Mr. Woolhouse, in the Newcastle Magazine.

weights on the funicular polygon. See Dr. Gregory's Mechanics, page 134; Whewell's Mechanics, page 34.

If W_1, W_2, &c.$=0$; then $\dfrac{\frac{1}{2}(\omega_1+\omega_2)}{\tan.\theta_1-\tan.\theta_2}=\dfrac{\frac{1}{2}(\omega_2+\omega_3)}{\tan.\theta_2-\tan.\theta_3}$.

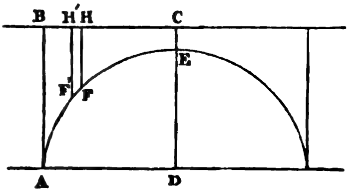
When the weights of the bars are also equal, we have $\tan.\theta_1-\tan.\theta_2=\tan.\theta_2-\tan.\theta_3$, &c...$=\tan.\theta_{n-1}-\tan.\theta_n$. Since the weights on the several joints are as the difference of the tangents of the angles of elevation, when these bars are indefinitely diminished, or the polygon becomes the curve of equilibration, we have ultimately the weight on any point proportional to the tangent of inclination to the horizon.[1]

[1] Or this may be shown from the principles of Coulomb, Mem. Pres. &c., tom. vii.: he there shows that if an arch rest in equilibrium on a base mn, it is necessary that the resultant of the two acting forces, viz., the horizontal force acting at AB, and the weight of the part A mn B acting vertically, shall be perpendicular to mn, and that it shall not fall without mn. Should the first condition be wanting, the arch will yield by sliding along mn; and if the second be wanting, it will have a rotatory motion about that extremity towards which the resultant falls.



Let W be the weight of the portion A mn B, F the horizontal force on AB, ι the inclination of the joint mn to the vertical; then if the weight of the part A mn B be represented by the vertical line pr, and the force F by the horizontal line pq, which meet in p, we have, since the resultant ps is perpendicular to mn, the triangles psq and mon similar;

From this property we may immediately derive one of the most useful cases of equilibration, viz. The extrados being a horizontal line, it is required to determine the intrados, on the supposition that the equilibrium is maintained by vertical pressures on the voussoirs. Let C be the origin of co-ordinates, $CH=x$, $HF=y$, $CE=a$; $CD=\alpha$, $AD=\beta$, ι the inclination of the tangent to the horizon, and draw $H'F'$ indefinitely near to HF. Now if we suppose the whole mass to act perpendicularly on the voussoirs, the differential of the weight of the column may be represented by $HF.HH'=ydx$, and since $\frac{dy}{dx}$ = tang. of inclination, we have from the above $\int y\,dx = c\,\frac{dy}{dx}$ or $ydx = c.\,d.\frac{dy}{dx}$; c being a constant quantity, which will be determined afterwards:



$$\therefore y\,dx^2 = c\,d^2y.$$

Multiplying by dy, and integrating, we have

$$\tfrac{1}{2}\,(y^2 + \text{const.})\,dx^2 = \tfrac{1}{2}\,c\,dy^2$$

$\therefore pr : rs\ (pq) :: mo : on$

$W : F :: \sin.\iota : \cos.\iota :: \tan.\iota : 1,$

$\therefore W = F.\tan.\iota$, consequently W varies as $\tan.\iota$, since the horizontal force is constant.

Also since mn is a normal, and $\tan.\iota = \frac{dy}{dx}$, we have

$$W = F.\,\frac{dy}{dx} \text{ or } W\,dx = F\,dy,$$

an equation to the catenary.

$$\text{or } c\left(\frac{dy}{dx}\right)^2 = y^2 + \text{const.}$$

But when $y=a$, $\frac{dy}{dx}$ vanishes,

$$\therefore \text{ the correct integral is } c\left(\frac{dy}{dx}\right)^2 = y^2 - a^2$$

$$\therefore \frac{dy}{dx} = \frac{\sqrt{y^2-a^2}}{\sqrt{c}}, \text{ or } dx = \sqrt{c}.\frac{dy}{\sqrt{y^2-a^2}},$$

the correct integral of which, Hall's Diff. and Int. Calculus, page 313, is

$$x = \sqrt{c} \cdot \left(\text{hyp. log. } \frac{y+\sqrt{y^2-a^2}}{a}\right)$$

$$\therefore \frac{x}{\sqrt{c}} = \text{hyp. log. } \frac{y+\sqrt{y^2-a^2}}{a}.$$

If e be the hyperbolic base, we have

$$e^{\frac{x}{\sqrt{c}}} = \frac{y+\sqrt{y^2-a^2}}{a}$$

$$e^{-\frac{x}{\sqrt{c}}} = \frac{a}{y+\sqrt{y^2-a^2}} = \frac{y-\sqrt{y^2-a^2}}{a}.$$

By addition we have

$$e^{\frac{x}{\sqrt{c}}} + e^{-\frac{x}{\sqrt{c}}} = \frac{2y}{a}$$

$$y = \frac{a}{2}\left(e^{\frac{x}{\sqrt{c}}} + e^{-\frac{x}{\sqrt{c}}}\right).$$

Differentiating,

$$\frac{dy}{dx} = \frac{a}{2\sqrt{c}}\left(e^{\frac{x}{\sqrt{c}}} - e^{-\frac{x}{\sqrt{c}}}\right),$$

which shows that the curve is convex towards the axis of x.

To find the constant c, we must observe that when F comes to A, $y=\mathrm{CD}=\alpha$, and $x=\mathrm{AD}=\beta$,

$$\therefore \beta=\sqrt{c}.\text{ hyp. log. }\frac{\alpha+\sqrt{\alpha^2-a^2}}{a}$$

$$\text{or }\sqrt{c}=\beta\div\text{hyp. log. }\frac{\alpha+\sqrt{\alpha^2-a^2}}{a}.$$

Hence by substitution we have

$$x=\beta.\text{ hyp. log. }\frac{y+\sqrt{y^2-a^2}}{a}\div\text{ hyp. log. }\frac{\alpha+\sqrt{\alpha^2-a^2}}{a}.$$

Also, from the above, another property may be derived, on which Dr. Hutton bases many of his calculations. If $\rho=$ radius of curvature, then we have

$$\rho=\frac{dz^3}{dyd^2x-dxd^2y},\text{ which becomes,}$$

if we suppose x to be the independent variable,

$$\rho=\frac{dz^3}{dxd^2y},$$

since the curve is convex to the axis of x,

and
$$cd.\frac{dy}{dx}=y\,dx$$

$$\therefore \frac{d^2y}{dx^2}=\frac{y}{c},$$

$$\text{but } dz=\sqrt{dx^2+dy^2}=dx\sqrt{1+\frac{dy^2}{dx^2}};$$

hence by substitution

$$\rho=\frac{c\left(1+\frac{dy^2}{dx^2}\right)^{\frac{3}{2}}}{y},$$

$$\text{but } \frac{dy}{dx} = \tan. \iota,$$

$$\therefore \rho = \frac{c \sec.^3 \iota}{y}.$$

Consequently the radius of curvature is proportional to $\sec.^3 \iota$ directly, and HF inversely, or HF is as $\sec.^3 \iota$ directly, and the radius of curvature inversely, c being constant.

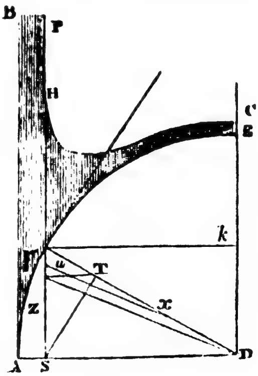
At the vertex E of any curve the inclination is nothing; therefore $\sec. \iota = 1$, and if the radius of curvature there be represented by ρ', the general expression for the height becomes $a = \frac{c}{\rho'}$ or $c = a\,\rho'$, which is the general value of c for any curve, in terms of the height of the crown and radius of curvature at that point: this substituted in the general expression, we have

$$\text{HF} = \frac{c}{\rho} \sec.^3 \iota = \frac{a\rho'}{\rho} \,.\, \sec.^3 \iota.$$

If the arc be the segment of a circle, then $\frac{\rho'}{\rho} = \frac{\rho}{\rho} = 1$,

$$\therefore \text{HF} = a \sec.^3 \iota.$$

This may be very simply calculated by logarithms, $\log. \text{HF} = \log. a + 3 \log. \sec. \iota$. Or we may give a geometrical construction as follows:



Draw the vertical line FS cutting the horizontal diameter in S, draw ST perpendicular to the radius DF, draw the horizontal line Tz, cutting the vertical in z, join Dz. Make Fu=CE, and draw $u\,x$ parallel to

zD; then FH must be equal to Fx, and by similar triangles,

$$FD : FS :: FS : FT = \frac{FS^2}{FD},$$

$$FS : FT :: FT : Fz = \frac{FS^4}{FD^2 . FS} = \frac{FS^3}{FD^2},$$

$$CE : FH :: Fu : Fx :: Fz : FD :: \frac{FS^3}{FD^2} : FD :: FS^3 : FD^3 :: Dk^3 : DE^3.*$$

The curve CHP runs up to an infinite height above the spring of the arch, and this must evidently be the case with every curve that springs at right angles to the horizontal line.

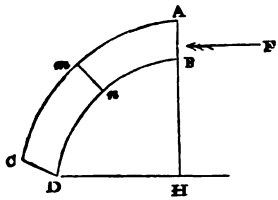
From the above it appears, that a semicircular arch cannot be put in equilibrio by building upon it, whatever may be its span or thickness at the crown; since the curve CHP runs up indefinitely, having AB for its assymptote; and therefore, according to the principles of equilibration, it is not adapted for a bridge which requires an outline nearly horizontal, except for about 30 or 40 degrees on each side of the vertex C.

From the general expression $HF = \frac{c}{\rho} \sec.^3 \iota$, and the equation of the curve, we may find either the extrados or intrados, whichever may be the given curve.

The celebrated experimental philosopher Coulomb,

* Dr. Gregory's "Mechanics," art. Arches.

to whom practical science is so deeply indebted, seems to have been the first who treated this important subject in a manner conformable to experiment and observation: he considers the equilibrium of arches successively on the hypothesis of joints perfectly polished, and on that where the friction of mortars or cements is taken into consideration. In the first place, he takes the part AB*nm* of the half arch ABCD as a body supported on an inclined plane, and gives, having respect to the effects of friction and cohesion, the limits between which the force



F acting horizontally at some point in the joint AB must fall, so that the arch may be prevented from sliding along the joint *m n*, either downwards in direction *m n*, or upwards in direction *nm*, or by rotation inwards round the point *n* of the joint *m n*, or outwards round the point *m*. The general expression for F being determined, he finds the maximum value of it, which will prevent the arch from sliding in direction *m n*, and the minimum value, which will cause it to slide in the opposite direction *n m*: he then observes that the limit of F, or the force which will be just sufficient to preserve the equilibrium, must be greater than the maximum and less than the minimum values so obtained.

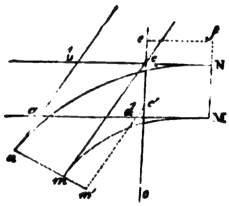
He then considers what the conditions of equilibrium must be, so that rotation cannot take place either round *m* or *n*, and finds in a similar way the maximum value of F, which will prevent rotation round the point *n*, and the minimum value that will cause it to move round *m*,

and shows that to maintain the equilibrium the force must be greater than the maximum and less than the minimum.

From this we have two superior and two inferior limits obtained for the force F, between which it must be found, so that the arch cannot slide either in the direction *m n* or *n m*, nor yet revolve round either of the points *m* or *n*. Gauthey, speaking of the analysis of Coulomb, observes that it leaves nothing wanting to make it coincide with that to which we are conducted by the latest experiments on the strength of arches.

That distinguished individual, the Rev. H. Moseley, Professor of Natural Philosophy in King's College, London, has, in two excellent papers in the Transactions of the Cambridge Philosophical Society, developed with great ability a theory for the equilibrium of the arch, on principles somewhat different from those of Coulomb, and has deduced expressions which accord, not only with a great many experiments made by himself, but also with those of Gauthey and Professor Robison. As this learned gentleman has kindly consented to give us some account of this theory, which appears to include that of Coulomb, we shall, for the present, defer entering into his views on this subject, considering that he will be better able to explain them himself; but we may observe, that it is our intention, in this work, to enter fully into the merits of both, and give as clear an exposition of each as we possibly can. Professor Moseley's Paper will be found in a future portion of this work.

We have, in the preceding pages, given a general outline of the celebrated theory of equilibration, which has long been considered by many eminent writers as one of the most delicate researches in applied mathematics, and indeed it seems to be almost too delicate for practical purposes, being based on suppositions which cannot hold good in practice; neither is the equilibrium sufficiently stable except under certain circumstances; and to ensure the stability of this equilibrium some writers have shown that the vertical line through the centre of gravity of the part *mn*NM should fall within the parallelogram *abcd*, formed by perpendiculars drawn at the extremities of the lines MN, *mn*, for otherwise the resultant of the two forces, viz., the weight of the part *mn*NM, and the horizontal thrust, would fall without *nm* where the arch rests, and from no point of that vertical could there be drawn two perpendiculars, the one to *mn* and the other to MN, so as to fall within both; consequently the part *mn*NM could not be sustained by those two joints or supports.



At the extreme point *c*, if the vertical line *ce* should pass through the centre of gravity, it is evident that from no other point in this line except *c* could there be drawn two perpendiculars, one to *mn*, and the other to MN; for if from any point *c′* below *c*, *c′m′* be drawn, the point *m′* will fall on *nm* produced, and if from any point *e* in this line above *c*, *ef* be drawn, it will fall in MN produced; *c* is therefore the utmost limit from

which the vertical line passing through the centre of gravity can be drawn for the equilibrium to have place, and it is clear that this is but a very unstable equilibrium, for the slightest force would derange it by throwing the vertical line beyond the required limits.

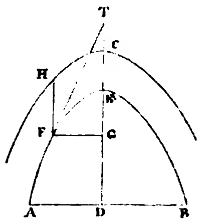
We shall now give a few examples to show how the extrados may be determined when the intrados is the given curve.

The intrados being a Parabola, required the extrados.

Let E be the origin, $EG = x$, $FG = y$, then

$$\rho = \frac{(m+4x)^{\frac{3}{2}}}{2\sqrt{m}} \text{ and } \rho' = \tfrac{1}{2}\, m \text{ and sec. } \iota = \frac{FT}{FG}$$

ρ and ρ' being the radii of curvature at F and E as before, and since $y^2 = m\,x$, and $GT = 2\,x$, we have sec. $\iota = \sqrt{\left(\frac{m+4\,x}{m}\right)}$; and by page 13,



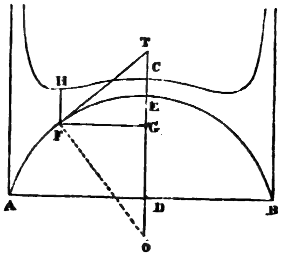
$$HF = \frac{a\,\rho'}{\rho} \text{ sec.}^3 \iota = a\,\frac{\frac{1}{2}\,m\,.\,2\,m^{\frac{1}{2}}}{(m+4\,x)^{\frac{3}{2}}}\,.\,\frac{(m+4\,x)^{\frac{3}{2}}}{m^{\frac{3}{2}}}. = a.$$

Hence it follows that the extrados is also a parabola equal to the intrados, and every where vertically equidistant from it.

To find the thickness over any point of an Elliptic arch.

Retaining the same notation for x, y, and a, let the semi-axis major $= b$, the semi-axis minor $= c$, and $FO = h$.

$DE^2 : AD^2 :: DG : OG$



$c^2 : b^2 :: c-x : \frac{b^2}{c^2}(c-x) = OG$

$\text{sec.}\,\iota = \frac{FO}{GO} = \frac{hc^2}{b^2(c-x)},\ \rho = \frac{4h^3}{m^2},$

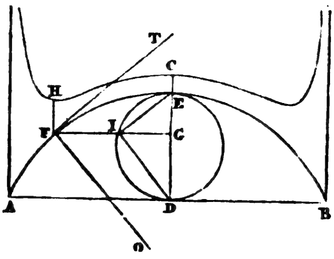
m being the parameter to $DE = \frac{2b^2}{c}$.

$$\therefore \rho = \frac{c^2 h^3}{b^4} \text{ also } \rho' = \frac{b^2}{c}$$

$$HF = \frac{a\rho'}{\rho}\,\text{sec.}^3\iota = \frac{ab^4}{c^2h^3} \cdot \frac{b^2}{c} \cdot \frac{c^6h^3}{b^6(c-x)^3} = \frac{ac^3}{(c-x)^3}.$$

For the Cycloid.

Let $2r$ be the diameter of the generating circle, then in this curve the following properties obtain, viz., the tangent FT is parallel to the corresponding chord EI, or the angle TFG = the angle EIG, also the radius of curvature at F is parallel to the supplemental chord ID, and equal to twice that chord, or $\rho = 2\,ID$,



$EI = \sqrt{2rx},\ GI = \sqrt{2rx - x^2},\ DI = \sqrt{4r^2 - 2rx},$

and therefore

$$\rho = 2\sqrt{4r^2 - 2rx}, \text{ and } \rho' = 2DE = 4r$$

$$\text{also sec.}\,\iota = \frac{EI}{GI} = \sqrt{\left(\frac{2rx}{2rx - x^2}\right)} = \sqrt{\left(\frac{2r}{2r - x}\right)}$$

$$\therefore \text{HF} = \frac{a\rho'}{\rho}\sec.^3\iota = \frac{a\,.\,4r}{2\sqrt{(4r^2-2rx)}}\cdot\frac{(2r)^{\frac{3}{2}}}{(2r-x)^{\frac{3}{2}}}$$

which by reduction becomes $\frac{4ar^2}{(2r-x)^2}$.

It may here be observed that in both the ellipse and the cycloid, the extrados is analogous to that of the circle, but somewhat flatter, and by computing values for FH the extrados is easily constructed by points.

We have throughout the whole of these articles considered the curve of equilibration to coincide exactly with the intrados, but this does not appear to be the most advantageous position for it to have. Dr. Thomas Young, in one of the ablest written articles which has appeared on this subject (see Napier's edition of the Encyclopædia Britannica), observes that "when the curve of equilibrium touches the intrados of an arch of any kind, the compression at the surface must be at least four times as great as if it remained at the middle of the arch-stones." This perhaps might be too high to fix its position when the bridge is constructed, since it is always found that arches settle either more or less after the centering is struck: the settlement then would throw the curve of equilibration above the middle of the arch-stones at the crown, but no allowance can be made for this, as it cannot be determined *à priori* what any arch will settle. If this curve coincides with the intrados, at the crown, before the centering is struck, it will undoubtedly afterwards take its place within the arch-stones, which is a

more advantageous position, so far as strength or durability of the arch is concerned.

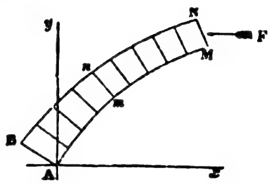
We shall now proceed to take a different view of this subject, and to show the conditions of equilibrium that must exist, in order that the arch may be so tied up that the voussoirs can neither slide upwards nor downwards, nor yet turn round either the upper or lower edges of their joints. This method of establishing the equilibrium of the arch has not been considered by any English writer till very lately, and even on the Continent, the place of its birth, it has not till within late years met with that attention which it so highly merits. The theory of Coulomb, which proceeds on this principle, has scarcely ever been noticed in this country; nay, the only English works we are acquainted with that inform us that Coulomb had at all written on the theory of arches are Cresswell's Venturoli, Dr. Gregory's edition of Dr. Hutton's course, and some of the writings of Professor Moseley.

On the equilibrium of an assemblage of voussoirs.

1. Let ABNM represent part of an arch, inclining at AB against a fixed plane, and supported at the other extremity by a force which is usually denominated the horizontal thrust. The form of this part of the arch is given by the curves of intrados and extrados, and the direction of the planes of the joints.

Now if we consider this part of the arch to be formed

by placing voussoirs successively upon each other, beginning at the fixed plane AB, it is clear that the first voussoirs placed against this plane would be supported merely by the effects of friction, and would continue to be



so supported until the inclination of the joints became so great as to cause them to slide;[1] it will then be necessary to apply to the joint MN of the last voussoir a force F, whose vertical and horizontal components may be represented by P and Q. This force ought to be sufficient to prevent the voussoirs from sliding downwards on the planes of the joints, and also to prevent them from turning on their lower edges; but it ought not to be so great as to cause the voussoirs to slide upwards on the joints, or to produce a rotary motion round their upper edges. We therefore see

[1] Mr. George Rennie, in a valuable paper in the Philosophical Transactions for 1829, states that "the granite voussoirs of the arches of the New London Bridge, having their beds well faced and dressed without mortar, generally commence sliding at angles from 33° to 34°. But with a bed of fresh and finely ground mortar interposed, the pressure on the centering commences at angles of from 25° to 26°. In other cases of arches, where sand-stones, such as Bramley Fall and Whitby, were employed, and their beds faced and dressed as usual, the angle of sliding was found to vary from 35° to 36°. But with mortar interposed, the angle generally varied from 33° to 34°.

"It results from these and other experiments that friction, by absorbing part of the horizontal thrust, is a most powerful assistant in maintaining the equilibrium of arches, and enables us to determine with something like precision the allowances due to theory."

generally that taking any joint mn whatever, the system of forces applied to the arch ABMN comprised in the components P and Q applied at the upper joint of the last voussoir, ought to be such that the action of the forces applied to the upper part mnNM cannot cause that part to slide on the plane of the joint mn, nor yet turn round either of the edges m or n.

Let x and y be the horizontal and vertical co-ordinates of the point m.

x', y', those of the point n.

θ the angle which the joint mn forms with the vertical.

z, the length of the joint mn.

a, b, the co-ordinates of the point M of the curve of intrados.

a', b', the co-ordinates of the point N of the curve of extrados.

W, ω, the vertical and horizontal components resulting from the weight of the portion mnNM of the arch.

α, β, the co-ordinates of the point c, where the components W and ω act.

f, the coefficient of friction, which is supposed to be proportional to the pressure.

r, the cohesive force of a unit of surface of the joint to prevent sliding.

R, the cohesive force of a unit of surface, which tends to prevent rotation.

T, the pressure in direction perpendicular to the joint $m n$.

2. To investigate the conditions relative to sliding on the joint *mn*, the force which tends to make the portion of the arch M*mn*N slide in the direction *nm*, is

$$(P+W)\cos.\theta$$

and the force which opposes this sliding is

$$(Q+\omega)\sin.\theta+f(P+W)\sin.\theta+f(Q+\omega)\cos.\theta+rz.$$

Now, that sliding may not take place in the direction *nm*, the latter must be greater than the former, or

$$P(1-f\tan.\theta)-Q(f+\tan.\theta) < -W(1-f\tan.\theta)$$
$$+\omega(f+\tan.\theta)+\frac{rz}{\cos.\theta}.$$

Also the force which tends to make the same portion slide in the opposite direction *mn* is

$$(Q+\omega)\sin.\theta$$

and the force which opposes this sliding is

$$(P+W)\cos.\theta+f(P+W)\sin.\theta+f(Q+\omega)\cos.\theta+rz.$$

Hence, that the sliding may not take place in the direction *mn*, we must have

$$-P(1+f\tan.\theta)+Q(\tan.\theta-f) < W(1+f\tan.\theta)$$
$$-\omega(\tan.\theta-f)+\frac{rz}{\cos.\theta}.$$

3. We must now proceed to establish the conditions that rotation may not take place round either of the points *m* or *n*.

Let us suppose, in the first place, that the portion of the arch *mn*NM tends to turn from the top to the

bottom on the edge or arris m, and that the force whose resultants are P and Q is applied at the point N, where it will have the least tendency to produce rotation round the edge m.

The moment of the forces which tend to make mnNM turn round m is

$$P\,(a'-x)+W\,(\alpha-x)$$

and the moment of the forces which oppose this rotation is

$$Q\,(b'-y)+\omega\,(\beta-y)\,;$$

but if we take into account the resistance from cohesion, we may adopt the theory of Mariotte and Leibnitz, viz., that the resistance to rupture which cohesion exerts at different points of the joint between m and n, will be proportional to the distances of those points from m.

Let an element of the joint mn be represented by $d\nu$, and ν its distance from the point m, the cohesion of this element will be $Rd\nu$, and the resistance exerted by this cohesion is $\frac{R}{z}\nu d\nu$, the moment of this resistance round the point m will be $\frac{R}{z}\nu^2 d\nu$; its integral taken between the limits of $\nu=o$, and $\nu=z$, or as it is usually expressed,

$$\int_0^z \frac{R}{z}\,\nu^2 d\nu=\tfrac{1}{3}\,Rz^2,$$

the moment from cohesion; hence the above becomes, taking into account the effects of cohesion,

$$Q\,(b'-y)+\omega\,(\beta-y)+\tfrac{1}{8}\,Rz^2;$$

therefore, in order that rotation may not take place round the point m, we must have

$$P\,(a'-x)-Q\,(b'-y)<-W\,(\alpha-x)+\omega\,(\beta-y)+\tfrac{1}{3}Rz^2.$$

Now, in the next place, if we consider the portion mnNM tends to turn on the edge n, and that the force whose resultants are P and Q is applied at M, where it will have the least tendency to produce this rotation; then the moment of the forces which tend to produce this motion is

$$Q\,(b-y')+\omega\,(\beta-y')$$

also the moment of the forces which oppose this motion is

$$P\,(a-x')+W\,(\alpha-x')+\tfrac{1}{3}Rz^2.$$

Therefore, in order that this rotation may not take place, we must have

$$-P\,(a-x')+Q\,(b-y')<W\,(\alpha-x')-\omega(\beta-y')+\tfrac{1}{3}Rz^2$$

The equilibrium of the portion mnNM requires, besides the condition mentioned in the last article, that the components P and Q should be such that these last conditions will also be satisfied for any joint mn.

Conversely, when the preceding conditions are satisfied for all the joints, the arch will necessarily remain in equilibrium.

4. The pressure in direction perpendicular to the joint mn, is

$$T=(P+W)\sin.\theta+(Q+\omega)\cos.\theta.$$

5. If in the preceding conditions relative to sliding we suppose the resistance from friction and cohesion to be nothing, we have, for a strict equilibrium,

$$P - Q \tan.\theta = -W + \omega \tan.\theta$$

$$\therefore \tan.\theta = \frac{P+W}{Q+\omega},$$

which shows that the resultant of the forces applied to the portion of the arch mnNM must be perpendicular to the joint mn.

6. Also the conditions relative to rotation become, when the friction and cohesion are not taken into account,

$$P(a'-x) - Q(b'-y) < -W(\alpha - x) + \omega(\beta - y)$$
$$-P(a-x') + Q(b-y') < W(\alpha - x') + \omega(\beta - y')$$

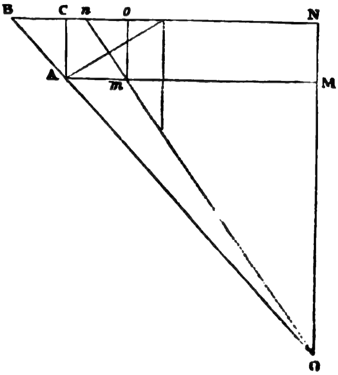
which show that the resultant of the forces applied to mnNM must fall between the points m and n.

7. By the preceding we see that in order that there may be an equilibrium of a system of voussoirs, it is necessary that the components P and Q should satisfy four inequalities, which are to be verified for all the joints of the arch. Hence there exists certain limits between which the values of P and Q must be found. If these conditions do not contradict each other, and if the values of the components P and Q be such as to satisfy them, the equilibrium can subsist in the proposed system of voussoirs; and if we conceive the last joint MN applied against a fixed plane as the first joint A B is, and the system submitted to the action of forces which are applied to the voussoirs, we may rest assured that motion will not ensue.

Plate Bande.

8. The plate bande has its extrados and intrados, two parallel straight lines, and differs from all other arches, inasmuch as the joints are not perpendicular to the intrados.

Let ABMN represent half the plate bande, AM$=a$, the thickness M N$=t$; the inclination of the extreme joint A B with the vertical $=\theta'$, the weight of a unit of mass$=\pi$, and retaining the same notation as in art. 1, the area of the trapezoid MmnN is equal the sum of the areas of the rectangle MNom and the triangle mno,



$$\text{area of rectangle} = (a-x)\,t$$
$$\text{area of triangle} = \tfrac{1}{2}\,t^2 \tan.\,\theta$$

$\therefore$ the weight of the trapezoid$=\pi\left\{(a-x)\,t+\frac{1}{2}\,t^2\tan.\,\theta\right\}$

Abstracting from the effects of friction and cohesion, and making, in art. 5, P$=0$, H$=0$,

$$W=\pi\left\{(a-x)\,t+\tfrac{1}{2}\,t^2\tan.\,\theta\right\};\ \text{we have}$$

$$\tan.\,\theta=\frac{2\pi\,(a-x)\,t}{2\,Q-\pi\,t^2}$$

which gives for the last joint AB

$$\tan.\,\theta'=\frac{2\,\pi\,a\,t}{2\,Q-\pi\,t^2}$$

hence $$Q=\frac{\pi(2at+t^2\tan.\theta')}{2\tan.\theta'}$$

$$\therefore \frac{\tan.\theta}{\tan.\theta'}=\frac{a-x}{a}.$$

This last equation shows that if the joints AB and *mn* were produced, they would intersect the vertical in the same point o; hence the condition of equilibrium established in art. 5 will be satisfied if all the joints produced shall pass through the same point.

9. The conditions of art. 6 become for the plate bande where $y=0$, $y'=t$, $b=0$, $b'=t$, $P=0$, $\omega=0$,

$$Qt > W(\alpha-x),$$
$$-Qt < W(\alpha-x'),$$

for any joint *mn*; but for the extreme joint AB we have

$Qt >$ moment of trapezoid round A,

$-Qt <$ moment of trapezoid round B.

The moment of the trapezoid round A is the difference of the moments of the triangle ABC, and the rectangle ACNM,

moment of rectangle $= \frac{1}{2}a\,.\,\pi a t$

. . . . . . triangle $= \frac{1}{3}t\,.\tan.\theta'.\frac{1}{2}\pi t^2\tan.\theta'$

$\therefore$ moment of trapezoid $= \frac{1}{2}\pi a^2 t-\frac{1}{6}\pi t^3.\tan.^2\theta'$.

But the moment of the trapezoid round B is equal to the sum of the moments of the triangle and rectangle.

Moment of rectangle $= (\frac{1}{2}a+t\,.\tan.\theta')\,at\pi$

. . . . . . triangle $= \frac{2}{3}t\,.\tan.\theta'.\frac{1}{2}\pi t^2.\tan.\theta'$

$\therefore$ moment of trapezoid round B is

$$(\tfrac{1}{2}a+t\,.\,\tan.\,\theta')\,at\pi+\tfrac{1}{3}\pi t^3.\,\tan.\,^2\theta'.$$

Hence the above conditions for the joint AB

$$Qt > \tfrac{1}{2}\pi a^2 t-\tfrac{1}{6}\pi t^3.\,\tan.\,^2\theta',$$

$$-Qt < \tfrac{1}{2}\pi at\,(a+2t\,.\,\tan.\,\theta')+\tfrac{1}{3}\pi t^3.\,\tan.\,^2\theta'.$$

The second inequality shows that the plate bande ABNM cannot turn round the upper arris B, since whatever positive value be given to Q, this condition will be satisfied.

The first inequality shows that ABNM cannot turn on the lower arris A, and it becomes, by substituting for Q its value from art. 8,

$$at > \tfrac{1}{2}(a^2-t^2)\tan.\,\theta'-\tfrac{1}{6}t^2\tan.^3\theta'$$

The extreme limit for θ' may be found by solving the equation,

$$\tan.^3\theta'-3\left(\frac{a^2-t^2}{t^2}\right)\tan.\,\theta'+\frac{6a}{t}=0.$$

If we take $\theta'=45°$, then since $\tan.\,45°=1$, the equation becomes, if we make $t=1$,

$$1-3\,(a^2-1)+6a=0$$

this solved gives $a=1+\sqrt{\tfrac{7}{3}}=2{\cdot}527$, or the whole span 5·054, which shows that the utmost limit to which the span can be extended is about five times its thickness; but that the equilibrium may be stable we must have $a<2{\cdot}527$, or the whole span $<$ 5·054.

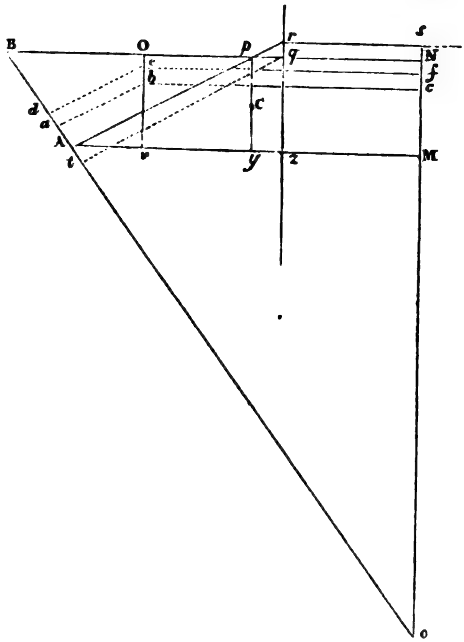
If $\theta'=30°$, then $\tan.\,\theta'=\dfrac{1}{\sqrt{3}}$ and the equation becomes

$$\frac{1}{3\sqrt{3}}-3\,(a^2-1)\,.\,\frac{1}{\sqrt{3}}+6a=0$$

which solved gives $a=3{\cdot}76$, or the whole span 7·52;

here the utmost limit gives about seven and a-half times the thickness for the extent of the span.

10. If a vertical line *py* be drawn through the centre of gravity, and from *p*, *p*A be drawn, then if AB be drawn perpendicular to *p*A, it will represent the extreme limit for the inclination of the joint; for we can draw from no other point in the line *py*, except *p*, two lines, the one perpendicular to MN, and the other to AB. Draw any two lines, O*v* and *rqz*, one on each side of the vertical through the centre of gravity; from A produce A*p* to *r*, then the other perpendicular *rs* will evidently fall above N; and if from the point *q* the line *qt* be drawn parallel to A*pr*, it will fall below A, or in BA produced, and from any other point in the line *qz* below *q*, the perpendicular will fall still farther below A; also from any point above *r* in this line, the perpendicular will fall farther above N:



but, on the contrary, in the line O*v*, we can draw from any point between O and A*p*, perpendiculars to AB and MN, such for instance as *ab*, *bc*, *de*, *ef*, &c. From this it follows that the equilibrium will be stable if the

centre of gravity should fall in any vertical line O*v*, within the triangle AB*p*; strict but easily deranged if it fall in *py*; and should it fall in any vertical line beyond this as *qz*, the arch cannot stand independent of friction and cohesion.

By art. 4 the pressure exercised perpendicularly on any joint *mn* is

$$T=(P+W)\sin.\theta+(Q+\omega)\cos.\theta,$$

which becomes for the plate bande where $P=0$, $\omega=0$

$$T=W\sin.\theta+Q\cos.\theta,$$

and for the last joint AB, by substituting for W and Q their values, viz., $W=\pi\,(at+\frac{1}{2}t^2.\tan.\theta')$ and

$$Q=\frac{\pi\,(2at+t^2.\tan.\theta')}{2\tan.\theta'}$$

we have

$$T=\pi\left(\frac{at}{\sin.\theta'}+\frac{t^2}{2\cos.\theta'}\right).$$

11. If the thickness of the plate bande be very small, compared with the span; then for the utmost limit, the equation $at-\frac{1}{2}(a^2-t^2)\tan.\theta'+\frac{1}{6}t^2.\tan.^3\theta'=0$ becomes, neglecting the quantities that involve t^2, which will be very small,

$$at-\tfrac{1}{2}a^2\tan.\theta'=0$$

$$\therefore\ \tan.\theta'=\frac{2t}{a}.$$

Therefore

$$Q=\frac{\pi at}{\tan.\theta'}=\tfrac{1}{2}\pi a^2;$$

also the expression for the perpendicular pressure on AB becomes

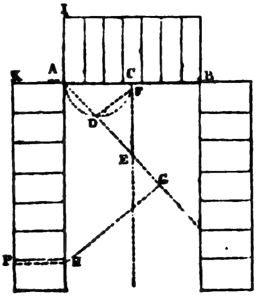
$$T=\frac{\pi at}{\sin.\theta'}$$

The following problem is taken from Professor Moseley's "Illustrations of Science," page 211.

"TO FIND THE GREATEST HEIGHT OF THE PIERS, OF A GIVEN WIDTH, WHICH WILL SUPPORT A STRAIGHT ARCH OF GIVEN DIMENSIONS.

12. Let AIB be the straight arch to be supported, and AK the given width of the piers.

Divide AB into two equal parts in C: upon AC describe a semicircle, and measure off AD equal to AK, so as to cut the circumference of this semicircle in D: produce AD, and let it intersect the vertical line through C in E: measure off EF equal to AI, and AG equal to AB: join DF, and draw GH parallel to DF; then AH will be the extreme height of the pier. Being of any less height, it will stand firmly; being of any greater, it will be overthrown."



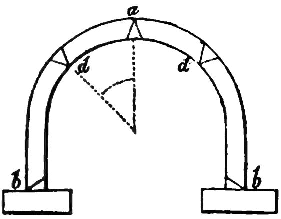
13. To establish the theory of arches, La Hire and other geometers supposed that they always break at points equally distant from the key-stone and the springings, and that the higher portion acts like a wedge against the joints of rupture, and tends to separate the lower portions; consequently, to obtain the thickness which the piers or abutments must have in order to resist this thrust, they endeavoured to find the pressure which is exerted perpendicular at one of the joints of rupture by making the moment with respect to the exterior edge

of the base of the pier equal to the moment of the half arch and its pier with respect to the same edge, and thus obtained an equation of equilibrium which gave the thickness of the pier.

The experiments of Danisy, and afterwards those of Boistard, have shown that arches may give way by sliding as La Hire supposes, but that the rupture more frequently occurs by means of rotation on the edges of the joint of rupture.

They have also proved that the position of the intermediate joints of rupture varies according as the force exerted by one part of the arch is greater than that exerted by the other.

14. If the force exerted by the upper portion be greater, that part tends to descend by means of the lower parts giving way, and the arch will break, as shown in the figure, that is to say, in five places, and the four portions of the arch turn round the edges b, d, a, d', b'; but if on the contrary the force exerted by the lower portions is the greater, the arch breaks as shown in the figure, page 58.



Arches, therefore, give way at the key-stone, at the springings, and at the intermediate points; but as it has been before observed, the position of the last points of rupture varies according as the force exerted by the upper or lower extremities is greater.

Coulomb, profiting by the experiments of Danisy, was the first that considered the theory of arches with respect both to the sliding of the voussoirs on each other, and

also rotation round the upper or lower edges of the joints. He showed that the theory of La Hire was insufficient; and first observed that an arch may break into four parts instead of three. (See Mémoires Présentées, &c. tome vii. pp. 381 and 382).

15. Since the time of Coulomb succeeding writers have contented themselves with developing his theory: among the most successful may be mentioned Colonel Audoy in No. 4, and Mr. Petit in No. 12, Mémorial de l'Officier du Génie. Also perhaps one of the clearest expositions of Coulomb's theory may be found in Navier's excellent work, entitled *Résumé des Leçons données à l'Ecole des Ponts et Chaussées.*[2] Garidel's *Tables des Poussées des Voûtes en Plein Cintre*, and *Mémoire sur la Stabilité des Voûtes*, by M. G. Lamé and E. Clapeyron, may be consulted with much advantage. These two latter emiment engineers gave a number of transcendental equations for determining the points of rupture, but they have given the investigation as regards rotation only, and experience proves that this kind of rupture is most to be dreaded.

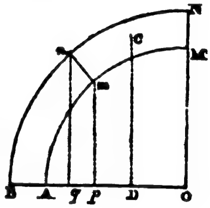
16. To proceed with the exposition of Coulomb's theory we may consider as given—the span, the rise, the curve of intrados, the height of the piers or abutments, the distribution of the weights which the arch ought to sustain, and the thickness of the arch at the key; this thickness is generally determined from the example of the most perfect constructions, similar to the one about

[2] To this admirable work we are greatly indebted, and consider that it should be used in every school that is at all interested in the progress of science applied to the arts.

to be projected. The equilibrium of the arch is maintained either by loading the parts more that tend to be raised, or by giving more thickness to these parts.

Let us suppose the arch to be divided into two equal parts at the key, the action of the weights supported by the arch exercises a pressure perpendicular to this joint between the two halves; we may therefore suppress one half of the arch, and replace it by a horizontal force equal to the pressure it exercises.

17. Let ABNM represent a semi-arch, and *mn* any joint where the rupture may be supposed to take place. Let OM and ON$=b$, b'; x, y the co-ordinates of the point m; x', y' those of the point n; z the length of the joint $m\,n$; θ the angle which this joint forms with the vertical; $a=$AD the distance of the point A from the vertical passing through the centre of gravity of this portion.



T the normal pressure on the joint *mn*, and for the rest the same notation as in art. 1.

18. Supposing that the rupture of the arch takes place by the sliding of the voussoirs along the planes of the joints, we have by art. 2

$$F=\frac{W(\cos.\theta-f\sin.\theta)-rz}{\sin.\theta+f\cos.\theta}.* \quad . \quad . \quad (1)$$

The values of F in this equation must be calculated for all the joints in the semi-arch, since by varying the

* The friction which is proportional to the perpendicular pressure on the joint will be

$$f(F\cos.\theta+W\sin.\theta)$$

and the cohesion being proportional to the length of the joint, is rz.

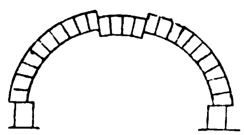
angle θ, we will have different values for F. The greatest of these values ought to be taken for the horizontal thrust, or the pressure which the two halves of the arch exercise against each other at the key.

The force F, which would be sufficient to cause the portion MNnm of the arch to slide upwards in the direction mn, may be thus expressed:

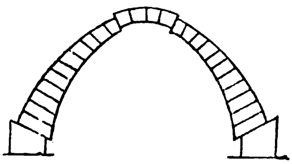
$$F = \frac{W(\cos.\theta + f\sin.\theta) + rz}{\sin.\theta - f\cos.\theta} \quad . \quad . \quad . \quad . \quad (2)$$

The values of F being calculated in the same way for all the joints in the semi-arch, the least value must be greater than the horizontal thrust, or the equilibrium requires that the maximum of equation (1) must be less than the minimum of equation (2).

19. According to the forms and proportions generally given to arches, the joints of rupture which will give the maximum of equation (1) will be found in the haunches of the arch. The joint which gives the minimum value of equation (2) is near the springings; here the arch has a tendency to give way, as in the annexed figure, the upper parts sliding downwards, and thereby forcing the lower parts upwards along the springing line.



It may also happen that the joint which gives the maximum of equation (1) may be at or near the springing line, whilst that which corresponds to the minimum of equation (2) is near the crown; then the arch will be apt to give way as in the annexed figure.



20. Now that the rupture cannot take place by the voussoirs turning on the upper or lower edges of the joints, by art. 1, the horizontal force F, applied at the point N, necessary to prevent the portion *mn*NM of the arch from turning from the top to the bottom on the lower edge or arris *m*, may be thus expressed:

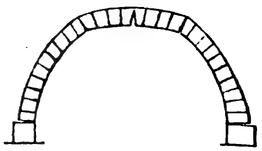
$$F = \frac{W(\alpha - x) - \frac{1}{3} R z^2}{b' - y} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The values of (3) being calculated for all the joints in the semi-arch, the greatest of these values ought to be taken for the horizontal thrust. Also, the horizontal force F applied at N, which would be sufficient to cause the portion *mn*NM to turn upon the upper arris *n*, may be expressed by the equation

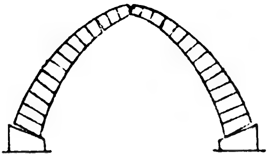
$$F = \frac{W(\alpha - x') + \frac{1}{3} R z^2}{b' - y'} \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

The values of (4) being also calculated for all the joints in the semi-arch, the least value must be taken greater than the horizontal thrust; that is, the equilibrium requires that the minimum of equation (4) must be greater than the maximum of equation (3).

From the above the maximum of equation (3) is given by a joint of rupture near the key, and the minimum of equation (4) given by a joint of rupture near the springings. Then the arch is apt to break, as represented in the annexed figure, the upper parts turning inwards round the arris *m*, forcing the lower parts of the arch to turn round the exterior arris of the joints near the springings.



21. But, at page 80, "Papers on Bridges," we have seen



that the arch may give way, as in the figure, the lower parts turning inwards, and thereby forcing the upper parts outwards. For the equilibrium for this kind of rupture, the force F applied at M, which would prevent the portion of the arch *mn*NM from turning on the edge or arris *m*, may be thus expressed:

$$F = \frac{W\,(a - x) - \frac{1}{3}\,R\,z^2}{b - y} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

The maximum of this equation must be taken for the horizontal thrust of the arch.

Also the force F applied at M, figure, page 36, which would cause the same portion to turn on the arris *n*, is

$$F = \frac{W\,(a - x') + \frac{1}{3}\,R\,z^2}{b - y'} \quad . \quad . \quad . \quad . \quad . \quad (6)$$

For the equilibrium in this case, the maximum of equation (5) must be less than the minimum of equation (6).

22. In art. 18 we have supposed the rupture to take place only by sliding, and in arts. 20 and 21, that it should be effected by rotation. The most general case is where the arch is apt to give way as is represented in the figure, page 38; it also sometimes happens that whilst the upper parts, as here represented, are descending, they force the lower parts outwards, as in the figure, page 37. It is clear that to prevent this motion the maximum of the expression equation (3) must be less than the minimum of (2).

For every possible combination that can take place, we must, in the first instance, have the equations (1) and (3), throughout the whole arch, less than the equations

(2) and (4); and in the next place, the equations (1) and (5) less than (2) and (6).

23. When the arch is built or supported on piers or abutments the preceding formulæ may be here applied, if we consider the piers or abutments to form part of the arch; but this supposes that the stones or voussoirs of the piers are sufficiently long to extend throughout the whole thickness, for the theory here treated of is founded on the hypothesis that the arch can only give way at the joints, either by sliding on their planes or turning on the edges.

24. From the preceding we may proceed to give the conditions of equilibrium in every possible case. 1st. To find the horizontal thrust F applied at N by supposing different positions of the joint of rupture mn, in the haunches of the arch, and stopping at the position for which the expression art. 21

$$F = \frac{W(\alpha - x) - \frac{1}{8} R z^2}{b' - y},$$

or by neglecting the effects of cohesion,

$$F = \frac{W(\alpha - x)}{b' - y}$$

gives the greatest value.

We must next ascertain whether the horizontal thrust thus determined be sufficient to cause either the whole or a part of the semi-arch ABNM to turn on the exterior arris. Now it is clear that the horizontal thrust can more easily force the whole semi-arch to yield, either by sliding or rotation, than it can a part of that semi-arch, where the inclination of the planes of the joints to the horizon is greater.

For the equilibrium to exist, supposing the rupture in the first joint AB, we must see that the expression

$$F=\frac{W(\alpha-x')+\frac{1}{3}Rz^2}{b'-y'},$$

or neglecting the effects of cohesion,

$$F=\frac{W(\alpha-x')}{b'-y'},$$

calculated for this joint, is greater than the horizontal thrust.

We must also see whether the horizontal thrust can make either a part or the whole of the semi-arch slide on the planes of the joints; that is, we must show that the expression

$$F=\frac{W(\cos.\theta+f\sin.\theta)+rz}{\sin.\theta-f\cos.\theta},$$

or neglecting cohesion,

$$F=\frac{W(\cos.\theta+f\sin.\theta)}{\sin.\theta-f\cos.\theta},$$

calculated for any joint whatever near the springings, is greater than the horizontal thrust. If this joint were horizontal, the preceding expression would become

$$F=Wf+rz,$$

or neglecting cohesion,

$$F=Wf.$$

25. In the note, page 88, "Papers on Bridges," we observed that the theory there given gave the same results as far as rotation was concerned as the theory of Coulomb, but that the latter was much simpler in form. We shall now reproduce the formulæ given in the Papers from that of Coulomb.

Equation (3) page 38, $F = \frac{W(a-x)}{b'-y}$ neglecting cohesion. Here $a-x = DP$, the distance of a vertical through the centre of gravity of the portion mnNM from D, and $b'-y = EQ$, using the notation given in the Papers, we have

$$Q = \mu \frac{DP}{EQ}.$$

Now by equation (4) the force Q, applied at E to turn the whole semi-arch round the arris K, may be thus determined.

The moment of the upper portion of the arch round K is μ (DP+KR).

The moment of the lower portion is ν. KS, the sum of these moments is the whole moment of the weight of the semi-arch round the point or arris K.

Also the moment of Q round K=Q.KX=Q (EQ+KU). Now in order to have a strict equilibrium these moments must be equal

$$Q\,(EQ+KU) = \mu\,(DP+KR) + \nu.\,KS$$

$$\therefore\ Q = \frac{\mu\,(DP+KR) + \nu.\,KS}{EQ+KU}.$$

Hence to ensure the stability we must have

$$\mu \frac{DP}{EQ} < \frac{\mu\,(DP+KR) + \nu.KS}{EQ+KU},$$

or by reduction

$$\mu \frac{DP \cdot KU}{EQ} < \mu.\,KR + \nu.KS,$$

but the triangles DMP and DEQ are similar

$$DP : PM\ (=FQ) :: DQ : EQ$$

$$\therefore \mathrm{DP} = \frac{\mathrm{DQ} \cdot \mathrm{FQ}}{\mathrm{EQ}};$$

this substituted in the above gives

$$\mu . \frac{\mathrm{FQ}}{\mathrm{EQ}} \cdot \frac{\mathrm{DQ}}{\mathrm{EQ}} . \mathrm{KU} < \mu . \mathrm{KR} + \nu . \mathrm{KS},$$

which is the same expression as given at page 84, "Papers on Bridges."

From this we see clearly the superiority of the theory of Coulomb over the complicated theory of rotatory levers, having deduced the same expression from the most simple and evident principles; besides, as we have before observed, the former is more general than the latter, as it takes into account the tendency of the voussoirs to slide on the planes of their joints.

26. M.M. Lamé and Clapeyron find the joint of rupture by making $\frac{m\,a}{h}$ a maximum, where m is the mass above that joint, a the distance between the point of rupture, and the vertical passing through the centre of gravity and $h = \mathrm{E\,Q}$, (see figure, page 81, "Papers on Bridges:") they give

$$\mathrm{H}\left(\frac{\mathrm{M\,A}}{\mathrm{H}} - \frac{m\,a}{h}\right),$$

a maximum where H, M, and A are the corresponding quantities to the above, taking both the arch and its piers into consideration; and since H and $\frac{\mathrm{M\,A}}{\mathrm{H}}$ are constant quantities, the above will be a maximum when $\frac{m\,a}{h}$ is a maximum. For semicircular arches with parallel extrados let R and r be the radii of bases of the cylin-

ders of extrados and intrados, $r\theta$ the arc between the middle of the key-stone and the joint of rupture, then $h=$ $R-r\cos.\theta$; $m=\frac{R^2-r^2}{2}.\theta$ and $a=r\sin.\theta-x$, x being the distance of the centre of gravity of the mass m from the vertical passing through the middle of the key. Let us conceive the mass m to be divided into infinitely small elements by planes passing through the common axis of the cylinders, making between them the constant angle $d\theta$, the centre of gravity of each of the equal elements will be distant from the axis of the cylinders by a constant quantity r', which by the property of the centre of gravity is thus determined,

$$\frac{R^2 d\theta}{2}.\frac{2R}{3}-\frac{r^2 d\theta}{2}.\frac{2r}{3}=\left(\frac{R^2 d\theta}{2}-\frac{r^2 d\theta}{2}\right)r',$$

$$\text{whence } r'=\tfrac{2}{3}\frac{(R^3-r^3)}{(R^2-r^2)};$$

r' being known we must determine x,

$$x\left(\frac{R^2-r^2}{2}\right)\theta=\int r'\sin.\theta\left(\frac{R^2-r^2}{2}\right)d\theta,$$

$\therefore\ \theta x=r'(A-\cos.\theta)$, the value of the constant A is unity, for the integral ought to vanish when $\theta=0$; we have then

$$a=r\sin.\theta-\frac{2(R^3-r^3)}{3(R^2-r^2)},$$

and by substituting the values of m, a, and h in $\frac{ma}{h}$, we have

$$\frac{ma}{h}=\frac{\frac{(R^2-r^2)}{2}r\theta\sin.\theta-\frac{(R^3-r^3)}{3}(1-\cos.\theta)}{R-r\cos.\theta}, \ldots \quad (1)$$

which must be a maximum.

Putting u for $\frac{m\,a}{h}$, and differentiating for θ, we have

$$\frac{d\,u}{d\,\theta}=\frac{\left\{\left(\frac{R^2-r^2}{2}\right)r(\sin.\theta+\theta\cos.\theta)-\left(\frac{R^3-r^3}{3}\right)\sin.\theta\right\}\left\{R-r\cos.\theta\right\}-\left\{\left(\frac{R^2-r^2}{2}\right)r\,\theta\sin.\theta-\left(\frac{R^3-r^3}{3}\right)(1-\cos.\theta)\right\}r\sin.\theta}{(R-r\cos.\theta)^2}=0$$

$$\therefore\left(\frac{R^2-r^2}{2}\right)r\left\{(\sin.\theta+\theta\cos.\theta)\,(R-r\cos.\theta)-r\,\theta\sin.^2\theta\right\}=\left(\frac{R^3-r^3}{3}\right)\left\{\sin.\theta\,(R-r\cos.\theta)-(1-\cos.\theta)\,r\sin.\theta\right\}$$

$$\left(\frac{R^2-r^2}{2}\right)r\left\{\sin.\theta\,(R-r\cos.\theta)+R\,\theta\cos.\theta-r\,\theta\cos.^2\theta-r\,\theta\sin.^2\theta\right\}=\left(\frac{R^3-r^3}{3}\right)\sin.\theta\,(R-r)$$

$$R-r\cos.\theta-\frac{\theta}{\sin.\theta}(r-R\cos.\theta)=\frac{2\,(R^3-r^3)}{3\,(R+r)r}$$

$$R-r\cos.\theta-z\,(r-R\cos.\theta)=\frac{2\,(R^3-r^3)}{3\,(R+r)\,r},\ \text{putting } z \text{ for } \frac{\theta}{\sin.\theta}, \quad \ldots\ldots\ldots\ldots \quad (2)$$

which has for its root the arc θ, corresponding to the joints or arris of rupture.

From the form of this equation we see that the position of the point of rupture depends on the relation of the radii R and r, or, which is the same, on the relation of the thickness of the arch to the diameter.

This equation, being transcendental, can only be solved by approximation. In the following table we have the different arcs θ from 45° to 60° with the corresponding values of sin. θ, cos. θ, and $z=\dfrac{\theta}{\sin.\theta}$.

Values of θ.		Values of sin. θ.	Values of cos. θ	Value of $z=\dfrac{\theta}{\sin.\theta}$
45°	or 0·7854	0·7071	0·7071	1·1133
46	0·80285	0·7193	0·6947	1·1161
47	0·8203	0·73134	0·6820	1·1216
48	0·83776	0·7431	0·6691	1·1273
49	0·8552	0·7547	0·65606	1·1332
50	0·87266	0·7660	0·6428	1·1392
51	0·8901	0·7771	0·6293	1·1453
52	0·9076	0·7880	0·6157	1·1517
53	0·9250	0·7986	0·6018	1·1582
54	0·9425	0·8090	0·5878	1·1649
55	0·9599	0·8191	0·5736	1·1719
56	0·9774	0·8290	0·5592	1·1790
57	0·9948	0·8387	0·5446	1·1862
58	1·0123	0·8480	0·5299	1·1937
59	1·0297	0·8572	0·5150	1·2014
60	1·0472	0·8660	0·5000	1·2090

The radius is always supposed equal to unity.

Let us take for an example the case of an arch whose thickness is $\frac{1}{16}$th of its diameter, then $\dfrac{R}{r}=\dfrac{9}{8}$, and the equation (2) becomes

$$(9-8\cos.\theta)-z\,(8-9\cos.\theta)=1{\cdot}0637 \quad . \; . \; . \quad (3)$$

Suppose $\theta=45°$, and we get 1·5218, which is too large; also if $\theta=0$, we have 2, which is also too great: hence we conclude that the angle of rupture is between 45° and 90°. If $\theta=60$, we get a quantity ·7686, too small in the left hand member of equation (3); therefore the real value is between 44° and 60°.

As the two preceding suppositions have given results which differ from the second part of the equation (3) of ·5 and ·3 respectively, the true value appears to be nearer 60° than 45°, and from the above relation it appears that θ is about 55°; this substituted gives 1·0859, which is still a little too great, which shows that θ is greater than 55°; and since $\theta = 56°$ gives a result 1·0282, a value which is too small, we may therefore easily find by interpolation $\theta = 55° \ 23'$; for taking the difference for 1° or 60′, and the difference between result for 50°, and the second member of equation (3), we have

$$\cdot 0577 : 60 : : \cdot 0222$$

$$\begin{array}{r} 60 \\ \hline \cdot 0577)\ 1\cdot 3320\ (23' \\ 1\ 154 \\ \hline 1780 \\ 1731 \\ \hline 49 \end{array}$$

If $\theta = 55° \ 23'$ be substituted, it will give in the first member of equation (3) 1·064, which differs from the second member only by the small quantity ·0003.

Thus the circular arch, whose thickness is constant and rise $\frac{1}{16}$th of the span, has the point of rupture situated 55° 93′ from the middle of the key-stone, or 34° 37′ from the springings.

27. The theory of Coulomb, as originally given by that

illustrious mathematician, and subsequently reproduced by Navier, which is nearly the same as we have here given, supposes a separate discussion of the conditions of equilibrium of each particular voussoir, and establishes the required maxima and minima by a comparison of the various different results thus obtained. In this form it supposes an immense labour of calculation; and, after all, it determines only the conditions of equilibrium of an existing structure, lending its aid but indirectly, and with difficulty, to the engineer who would determine the form and dimensions of a proposed structure so as best to secure its stability. The subsequent labours of M.M. Audoy, Lamé and Clapeyron, Persy, Petit, Poncelet and Garidel, have however given to this theory a new form and character, embracing in the discussion many conditions of the equilibrium of the arch which lay before beyond its limits, and greatly diminishing the labour of its calculations. From the account which our limits permitted to give of it, it will appear that this new developement of the theory of Coulomb consists in the determination in terms of the inclination of any joint of the arch to the vertical of a function expressing the value of that horizontal thrust, which, being applied to the summit of the key-stone, will just prevent the semi-arch from turning inwards upon that joint. By the theory of Coulomb the maximum of this function determines the position of the joint of rupture.

Professor Moseley has shown that his theory, founded upon the discussion of the lines of resistance and pres-

sure, (the former of which he has been the first to introduce in the theory of statics,) and developing itself by a wholly independent method of analysis, so far as it embraces the same elements of the discussion with the theory of Coulomb, necessarily leads to the same results; and it is a remarkable verification of the formulæ given by this able mathematician, and of those deduced from the theory of Coulomb by the eminent individuals whose names we have mentioned, that, proceeding with methods of analysis so remote and so difficult, they have arrived at formulæ, which, when they refer to the same circumstances of equilibrium, are identical.

The formulæ arrived at by the French mathematicians as stated by Garidel, and made by him the foundation of the tables which he has calculated with so much labour and ingenuity, are the following; the notation being made to correspond with that of Professor Moseley's Paper. In the case in which the load rests on the extrados, and the arch is a complete segment, so that $\Theta=0$:

Garidel. Tables de la Poussée des Voutes, p. 30. $$\frac{P}{r^2}=\frac{3\left\{(1+\alpha)^2-1\right\}\Psi\ \sin.\ \Psi-2\,(1-\cos.\ \Psi)\left\{1+\alpha)^3-1\right\}}{6\,(\alpha+1-\cos.\ \Psi)} \quad \ldots\ldots \quad (1)$$

Garidel, p. 31. $$\frac{P}{r^2}=\alpha\left\{\left(1+\frac{\alpha}{2}\right)\Psi\ \cot.\ \Psi-\alpha\left(\frac{1}{2}+\frac{\alpha}{3}\right)\right\} \quad \ldots\ldots \quad (2)$$

Petit. Mémorial de l' Officier du Génie, No. 12. $$\cos.\ \Psi+\frac{\Psi}{\sin.\ \Psi}-(1+\alpha)\,\Psi\ \cot.\Psi=1+\alpha-\frac{2}{3}\,\frac{(1+\alpha)^3-1}{\alpha+2} \quad \ldots\ldots \quad (3)$$

In the case of an arch where loading is bounded by a straight line inclined to the horizon at a given angle :

Garidel, p. 44. $$\left.\begin{array}{l} 0=-A+B\cos.\ \Psi-C\cos.^2\Psi+D\cos.^3\Psi+\dfrac{\Psi}{\sin.\ \Psi}-(1+\lambda)\,\Psi\ \cot.\ \Psi \\ \quad +E\sin.\ \Psi\,(1-\frac{2}{3}\sin.^2\Psi)-H\sin.\ \Psi\ \cos.\ \Psi \end{array}\right\} \quad \ldots\ldots \quad (4)$$

Where $A=\frac{2}{3}+(1+\beta)\,(1+\alpha)\,(1-\alpha^2)\,+\alpha^2\,(1+\frac{2}{3}\alpha)\,(1+\lambda)$

$B=1+2\,(1+\beta)\,(1+\alpha)\,(1-\alpha^2)\,(1+\lambda)$

$C=(1+\alpha)\{(1+\beta)\,(1-\alpha^2)+(1+\alpha)\,(1-2\,\alpha)\,(1+\lambda)\}$

$D=\frac{2}{3}\,(1+\alpha)^2\,(1-2\,\alpha)$

$F=(1+\alpha)^2\,(1-2\,\alpha)\ \tan.\ \iota$

$H=(1+\alpha)^2\,(1-2\,\alpha)\,(1+\lambda)\ \tan.\ \iota$

When the direction of the pressure P is through the bottom of the key-stone :

Garidel, p. 44. $$\left.\begin{array}{l} 0=-\left\{\frac{2}{3}+(1+\alpha)(1+\beta)(1-\alpha^2)+\alpha^2(1+\frac{2}{3}\alpha)\right\}+\left\{\frac{1}{3}+(1+\beta)\,(1+\alpha)\,(1-\alpha^2)-\alpha^2\,(1+\frac{2}{3}\alpha)\right\}\cos.\ \Psi \\ \quad +2\left\{\dfrac{2\,\alpha^3}{3}+\alpha^2-\frac{1}{3}\right\}\cos.^2\Psi+\dfrac{\Psi}{\sin.\ \Psi}+(1+\alpha)^2\,(1-2\,\alpha)\ \sin.\ \Psi\ (\frac{1}{3}-\frac{2}{3}\cos.\ \Psi)\ \tan.\ \iota \end{array}\right\} \quad (5)$$

Now, if in equation (7) of Professor Moseley's Paper, page 51, X and Y be taken$=0$, by which substitutions the more general case of equilibrium supposed in that equation will be reduced to the case of a complete arch of equal voussoirs without loading, and if this equation be then solved in respect to $\frac{P}{r^2}$, then equation (1) of the above will be reproduced. Equation (13) of Professor Moseley's Paper will, in like manner, give us equation (2) of the above. If in the above equations (3) and (4), ι be assumed$=0$, and λ taken$=\alpha$, the Professor's equations (19) and (20) will be obtained. These are the fundamental equations from which the Tables of Garidel are calculated; and they result alike from the theory of Coulomb and that of Professor Moseley.[2]

The discussion of the latter theory, however, embraces various elements which are not, we believe, to be found in any other.

It determines the conditions of equilibrium not only of the continuous segmental arch, but of the Gothic arch, and that under every variety of loading; not only for instance when the pressure of the load is vertical, but when its direction is inclined at any angle to the ho-

[2] M. Garidel has given the following approximate expressions for the angle of rupture and the thrust in the case of an unloaded arch with equal voussoirs; they are derived from the equations (1) and (2).

$$\Psi=57^\circ 293+\frac{49{\cdot}594\,(\alpha+1{\cdot}69043)\,(\alpha-0{\cdot}15371)\,(0{\cdot}99987-\alpha)}{(\alpha+2)\,(\alpha+0{\cdot}4597)}$$

$$\frac{P}{r^2}=\frac{0{\cdot}1532\,\alpha\,(\alpha+1{\cdot}7106)\,(1{\cdot}4565-\alpha)}{\alpha+0{\cdot}4597}$$

rizon: as an illustration of this fact, let it be observed that equations (25, 26, 27) determine the conditions of equilibrium of an arch which sustains, either by its extrados or intrados, the oblique pressure of a fluid. The equations of the Professor's Paper not only determine what are the conditions of the stability of an arch, under a given loading, but what loading will give certain conditions of stability. They enable us, for instance, so to load a segmental arch as to bring its points of rupture to any given distance from its springing; they also determine what load accumulated near the crown would cause the arch to fall by the descent of the crown, and what at the haunches would cause its fall by the elevation of the crown. In respect to the direction of the pressure upon the key-stone, these formulæ include, in common with the French formulæ, the case in which (the arch being constructed without cement) this direction may be supposed to be through the summit of the key-stone; they determine also the actual direction of this pressure when by the interposed cement a mathematical adjustment of the joints is brought about, and when this direction is therefore through some point which intervenes between the top and bottom of the key-stone,—a determination which, we believe, has not been attempted by any other author. It is, however, principally to be remarked in respect to the theories of Coulomb and Professor Moseley, that the latter is general, embracing every case of the equilibrium of a system of bodies in contact, and including the arch as a particular case. Of these various applications, (others of which are given by the learned Professor,) are

the buttress, the pier, the straight arch or plate bande, the embankment, &c.

The formulæ given by the French mathematicians for determining the width of the pier of a given height is deducible from Professor Moseley's equations, substituting in equation (4) the values of the horizontal thrust, the weight of the arch, and the quantity k, as determined by the subsequent equations, and then solving equation (4) for K.

28. We shall now proceed to give Petit's formulæ and tables for calculating the stability of arches.

At page 38 we have shown that if the length of the arm of the lever, with which the horizontal thrust F acts $=y$, and the weight of the portion mnNM$=$W, and ϕ the length of the arm at which it acts, then we have

$$\mathrm{F}\,.\,y=\mathrm{W}\,.\,\phi \text{ or } \mathrm{F}=\frac{\mathrm{W}\,\phi}{y}.$$

We are ignorant of the point m round which the rotation tends to take place, or in other words, we do not know the angle θ which the joint of rupture mn makes with the vertical; but since the force F must be such that it shall keep any portion whatever of the arch in equilibrium upon the portion beneath it, it must be equal the greatest value that $\frac{\mathrm{W}\times\phi}{y}$ admits by the variation of the point of rupture.

In order to obtain this value, we must assume a point of the arch, or the angle θ, which determines the position of the point, and find the corresponding value of F. We must next assume another value of θ, and

find another value of F, and proceed in this way until we have found the maximum value of F. (See page 38.)

We must take the moment of this last force with respect to the edge *d*, or the outer edge of the given piers, and then put this moment, equal to the moment (M) of the half arch and its pier, and this equation will give (*e*) the thickness of the pier. Thus, if F' be the maximum value of F, and L the arm at which it acts, we have the equation

$$\mathrm{M}=\mathrm{F'L}, \text{ which gives } e.$$

These are the two formulæ which M.M. Audoy and Petit have developed. The following are M. Petit's formulæ, relating to arches that are semicircular and have a parallel extrados.

Semicircular arches with parallel extrados.

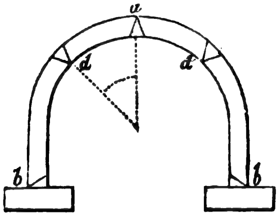
29. The formulæ for these arches are

$$\cos.\theta+(1-\mathrm{K}\cos.\theta)\frac{\theta}{\sin.\theta}=\mathrm{K}-\tfrac{4}{3}.\frac{\mathrm{K}^3-1}{\mathrm{K}+1} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$\mathrm{F'}=r^2\left\{\tfrac{1}{2}(\mathrm{K}^2-1)(1+\frac{\theta}{\sin.\theta}.\cos.\theta)-\tfrac{1}{3}(\mathrm{K}^3-1)\right\} \quad . \quad . \quad . \quad (2)$$

$$\frac{e}{r}=-\tfrac{1}{4}\pi(\mathrm{K}^2-1)\frac{r}{h}+$$

$$\sqrt{\tfrac{1}{16}\pi^2(\mathrm{K}^2-1)^2.\frac{r^2}{h^2}+2\left\{\mathrm{K}c+\tfrac{1}{3}(\mathrm{K}^3-1)-\tfrac{1}{4}\pi(\mathrm{K}^2-1)\right\}\frac{r}{h}+2c} \quad (3)$$



In the preceding formulæ K is the ratio of the radius

of the extrados to that of the intrados or $\frac{R}{r}$; θ the angle of rupture corresponding to the maximum thrust; F′ the maximum value of the thrust F; r the radius of the intrados; h the height of the pier or abutment of the arch; c the ratio of the maximum thrust, and the square of the radius or $c = \frac{F'}{r^2}$; π the ratio of the circumference to the diameter.

As an example, we may determine the necessary thickness for the piers or abutments of a semicircular arch with parallel extrados, where the radius of the intrados $r = 16{\cdot}4$ feet, and the radius of extrados R $= 20{\cdot}99$ feet, and the height of the piers or abutments 6·56 feet.

With the Tables I. and II. of M. Petit this problem can be easily solved. We must first find the ratio $K = \frac{R}{r} = \frac{20{\cdot}99}{16{\cdot}4} = 1{\cdot}28$. In Table I. we find the corresponding value of $\theta = 62°30'$, and Table II. gives the following equation to solve

$$\frac{e}{r} = -\,{\cdot}5014\cdot\frac{r}{h} + \sqrt{{\cdot}2520\cdot\frac{r^2}{h^2} + {\cdot}0801\cdot\frac{r}{h} + {\cdot}2738}$$

in order to determine the thickness of the abutments in the case of strict equilibrium.

The preceding equation is only equation (3) with the values $K = 1{\cdot}28$, $\pi = 3{\cdot}1416$, or $\frac{r}{h} = \frac{16{\cdot}4}{6{\cdot}56} = 2{\cdot}5$, and this equation gives $e = 2{\cdot}92$ for the case of strict equilibrium.

Without these tables we should first have supposed $\theta = 61°$ or $\theta = \frac{61 \times 3{\cdot}1416}{180} = 1{\cdot}06$, cos. $61° = {\cdot}4848$,

sin. $61° = \cdot 8746$; $K = 1 \cdot 28$; these numbers substituted in equation (1) we have $\cdot 9446 = \cdot 9592$; hence 61° does not satisfy the equation.

Next assume $\theta = 62°$, and, proceeding as before, we find $\cdot 95856 = \cdot 95920$; this value of θ therefore does not satisfy the equation.

Now assume $\theta = 63°$, and we find $\cdot 97093 = \cdot 95920$; here the first number is the greater, we therefore conclude that the real value of θ lies between 62° and 63°.

Taking the difference $\cdot 97093 - \cdot 95856 = \cdot 01237$, which corresponds to 1° or 60′, and also the difference $\cdot 95920 - \cdot 95856 = \cdot 00064$,

$$\cdot 01237 : 60' :: \cdot 00064 : x = 31';$$

hence $\theta = 62°\ 31'$; sin. $\theta = \cdot 8871$, cos. $\theta = \cdot 4615$, $\theta = \frac{62°\,31'}{90} \cdot \frac{3 \cdot 1416}{2} = 1 \cdot 091$. These values substituted in equation (2) give $c = \frac{F}{r^2} = \cdot 135$, nearly the same as the value given by Table I., whence we may find the maximum thrust $F' = \cdot 135 \cdot (16 \cdot 4)^2 = 36 \cdot 31$ feet; lastly, this value of c, and those of h, r, π, and K, substituted in equation (3), give the thickness e, as has been found above. In the case where h is infinite, the formula (3) would reduce to $\frac{e}{r} = \sqrt{2c}$, which gives for the limit of the thickness of the piers or abutments $e = 8 \cdot 528$ feet instead of $2 \cdot 92$.

By Colonel Audoy's method we may find the stability by multiplying the value of the thrust by $1 \cdot 9$, which gives the stability of arches calculated by the formula (3); by putting $1 \cdot 9\ c$ or $c + \cdot 9\ c$ instead of c, we have

$$\frac{e}{r} = -\tfrac{1}{4}\pi\,(K^2-1).\frac{r}{h} +$$
$$\sqrt{\left\{\tfrac{1}{16}\pi^2(K^2-1)^2.\frac{r^2}{h^2} + [1{\cdot}8.\,Kc + 2\,Kc + \tfrac{2}{3}\,(K^3-1) - \tfrac{1}{2}\pi\,(K^2-1)].\frac{r}{h} + 2\,c + 1{\cdot}8\,c\right\}}$$

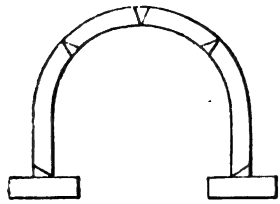
If we want to find the limit of thickness in the case of stability given by La Hire, the equation becomes $\frac{e}{r} = \sqrt{2\,c + 2 \times {\cdot}9\,c} = \sqrt{3{\cdot}8\,c} = {\cdot}72$, whence $e = {\cdot}72 \times 1{\cdot}64 = 11{\cdot}8$ feet.

The voussoirs tend to slide on the surfaces of the inferior joints; hence there results a thrust sometimes more powerful than that of rotation. In semicircular arches with a parallel extrados the maximum thrust due to sliding is $G = {\cdot}15304\,r^2\,(K^2-1) = F'$, G being the horizontal force capable of preventing the sliding of any voussoir whatever which tends to descend upon the lower surface. It is by these means that Petit has found the numbers which are given in the 5th column of Table I. Thus, in the example $c = \frac{F'}{r^2} = {\cdot}15304\,(K^2-1) = {\cdot}0977$, a thrust which is less than the thrust from rotation.

If the former thrust were the greater, we should use it to determine the thickness of the piers. By examining Table I. we find the values of c relative to sliding greater than those relative to rotation as far as $K = 1{\cdot}44$; thus, for those arches which give to K a value between $2{\cdot}732$ and $1{\cdot}44$, we must employ the values of c relative to sliding for determining the thickness of the abutments.

The following figure represents the rupture of the arch

when the force exerted by the lower portions is greater than that by the upper.



Semicircular arches with horizontal extrados.

30. The formulæ relative to these arches are

$$F=\frac{r^2 \sin.^2 \theta}{6(K-\cos.\theta)}\left\{K^2[6-3K-(3-2K)\cos.\theta]-\left(\frac{3\,\theta}{\sin.\,\theta}-\frac{1}{\cos.^2\frac{1}{2}\theta}\right)\right\} \quad (1)$$

and

$$\frac{e}{r}=-(K-\tfrac{1}{4}\pi)\frac{r}{h+K\,r}+\sqrt{(K-\tfrac{1}{4}\pi)^2\left(\frac{r}{h+K\,r}\right)^2+\left(2Kc-K+\frac{3\pi-4}{6}\right)\frac{r}{h+K\,r}+2\,c\,.\,\frac{h}{h+K\,r}} \quad (2)$$

The first gives the thrust due to rotation, and the second the thickness e of the piers.

And $G=r^2\,(\cdot 16391\,.\,K^2-\cdot 15206)$ (3), which gives the thrust due to the maximum of sliding. As an example, let us find the thickness of the abutments of an arch with an horizontal extrados, where the radius of intrados is $r=6\cdot 56$ feet, and radius $R=7\cdot 54$ feet, and the height of the piers $h=9\cdot 84$ feet.

We shall have $K=\frac{R}{r}=\frac{7\cdot 54}{6\cdot 56}=1\cdot 15$, and we might assume the angle of rupture $\theta=60^\circ$ and find the corresponding value of F, and then make $\theta=61^\circ$ and find the corresponding value of F by means of equation (1), and proceed in the same manner as at pages 55 and 56, until we found the maximum value of F; but, by Table III. we see that if $K=1\cdot 15$, the corresponding value of the

angle of rupture is $\theta = 64°$, and $\frac{F'}{r^2} = c = \cdot 11895$ in the case of rotation.

In the case of sliding we obtain by the same Table $\frac{F'}{r^2} = \cdot 06471$.

According to what we have before observed, we shall take the first value of c (it being greater than that due to sliding) to determine the thickness of the abutments; by substituting in (2) the values $K = 1 \cdot 15$, $\frac{F'}{r^2} = c = \cdot 11895$ and $\pi = 3 \cdot 1416$, we have

$$\frac{e}{r} = - \cdot 3646 \times \cdot 377 \sqrt{\cdot 01889 + \cdot 01044 + \cdot 13465},$$

hence $e = 1 \cdot 75$ feet for the case of strict equilibrium.

If h were infinite, the equation (2) would reduce to $\frac{e}{r} = \sqrt{2c} = \sqrt{2 \times \cdot 11895}$; hence $e = 3 \cdot 2$ feet for the limit of the thickness.

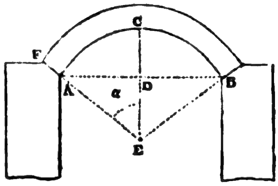
If we wanted the practical stability, we ought to substitute $1 \cdot 9\ c$ for c in equation (2). Similarly to obtain the limit of the thickness according to La Hire, we must substitute $1 \cdot 9\ c$ for c in formula $\frac{e}{r} = \sqrt{2\,c}$, which gives $e = r\sqrt{3 \cdot 8\, c} = 4 \cdot 34$ feet nearly.

The Table III. shows that for values of K less than $1 \cdot 35$ we must consider in our calculation the thrust due to rotation, since it exceeds that due to sliding, and we must consider the latter thrust if K be equal or greater than $1 \cdot 35$.

If in the above example we want the thrust due to sliding, we obtain

$$\frac{F'}{r^2} = \cdot 16391\ K^2 - \cdot 15206 = \cdot 0647 \therefore F' = 2 \cdot 785.$$

Arches in the form of a circular arc with parallel extrados.



31. Besides the data necessary for the two preceding cases we must have the span AB, which we denote by L, and the rise CD, denoted by f.

Having L and f given, we shall have, in order to determine the radius AE or r,

$$r = \frac{f}{2}\left(1 + \frac{L^2}{4f^2}\right)$$

and the angle AEC, which we denote by a, is given by the equation

$$\sin.\ a = \frac{\frac{L}{f}}{\frac{1}{4} \cdot \frac{L^2}{f^2} + 1}.$$

When the span L and the angle a are given, we shall have the radius r by the formula $r = \frac{L}{2 \sin.\ a}$, and f from $f = r\ (1 - \cos.\ a)$, r being known, and the thickness AF, we shall have EF$=$R, and therefore $\frac{R}{r} = K$. The Table I. relative to semicircular arches with a parallel extrados will give the angle of rupture θ. It may happen,—1st. That this angle of rupture of the proposed arch, considered as a complete semicircle, is smaller than a or half the angle at the centre; in this case the joint of rupture takes place between A and C, and the arch

ought to be considered relative to the horizontal thrust as a semicircular arch, and the Table I. will give the maximum thrust $F' = cr^2$ or $c = \frac{F'}{r^2}$. As for the thickness of the abutments it will be found by the equation

$$\frac{e}{r} = -\tfrac{1}{2} a \cdot \frac{r}{h}(K^2 - 1) + \sqrt{\left\{ \tfrac{1}{4} a^2 (K^2-1)^2 . \frac{r^2}{h^2} + 2\left[c(K - \cos. a) + \tfrac{1}{3}(K^3 - 1)(1 - \cos. a) - \tfrac{1}{2}(K^2 - 1) a \sin. a\right] \frac{r}{h} + 2c \right\}} \quad . \; . \; . \; . \; . \; . \; . \quad (4)$$

The limit of the thrust in the case of strict equilibrium is always given by $e = r\sqrt{2c} = \sqrt{2F'}$; that is, it is always equal to the square root of twice the horizontal thrust; and the stability, according to La Hire, is given by the equation

$$e = r\sqrt{2 \times 1{\cdot}9\, c} = r\sqrt{3{\cdot}8\, c}.$$

As an example, let us find the thickness of the abutments of an arch in the form of a circular arc with parallel extrados, where $a = 62°$, the thickness AF $= 2{\cdot}23$ feet, L $= 19{\cdot}68$ feet, and $h = 13{\cdot}12$ feet, we have

$$r = \frac{L}{2 \sin. a} = \frac{19{\cdot}68}{2 \times {\cdot}8829} = 11{\cdot}145 : f = r(1 - \cos. a) = 11{\cdot}145(1 - {\cdot}4695)$$
$$= 5{\cdot}9124;\ R = 11{\cdot}145 + 2{\cdot}23 = 13{\cdot}375, \frac{R}{r} = \frac{13{\cdot}375}{11{\cdot}145} = 1{\cdot}2 = K,$$

and to this value of K corresponds the angle $\theta = 59° \, 41'$ (Table I.), and $c = {\cdot}1114$, a value which is greater than the value of c due to sliding; we shall therefore employ $c = {\cdot}1114$, in order to find the thickness of the abutments. We shall also have $a = \frac{62}{70} \cdot \frac{3{\cdot}1416}{2} = 1{\cdot}0821$; all these values substituted in equation (4) give

$$\frac{e}{r} = -{\cdot}20223 + \sqrt{{\cdot}0409 + 2\left\{{\cdot}7305 . c + {\cdot}1287 - {\cdot}2101\right\} {\cdot}8425 + 2c},$$

whence $e = 11 \cdot 145 \times \cdot 31 = 3 \cdot 45$ when the equilibrium is strict.

The limit of the thickness $e = r\sqrt{2c} = 11 \cdot 145 \times \sqrt{\cdot 2228} = 5 \cdot 25$ feet.

For the stability according to La Hire we must put $1 \cdot 9\, c$ for c in the above formula, and we shall have

$$\frac{e}{r} = -\cdot 20223 + \sqrt{0.409 + 2\left\{\cdot 7305 \times 1 \cdot 9\, c + \cdot 1287 - \cdot 2101\right\} \times \cdot 8495 + 2 \times 1 \cdot 9\, c},$$

whence $e = r \times \cdot 56 = 6 \cdot 23$ feet, and the limit $e = r\sqrt{2 \times 19\, c} = 7 \cdot 25$ feet.

It may also happen that the angle of rupture of the proposed arch, considered as a semicircle, is greater than half the angle at the centre or a, which usually occurs in practice; then the rupture takes place at the springings, and the thrust is given in this case by the equation

$$c = \frac{F'}{r^2} = \frac{\frac{1}{2}(K^2 - 1)\, a \sin.\, a\, r^3 - \frac{1}{3}(K^3 - 1)(1 - \cos.\, a)\, {}^3r}{K - \cos.\, a}.$$

Thus L and f being given we have the ratio $\frac{L}{f}$, and the formula $\sin.\, a = \dfrac{\frac{L}{f}}{\frac{1}{4} \cdot \frac{L^2}{f^2} + 1}$ gives a; having a $\sin.\, a$, $\cos.\, a$, K and r, we obtain c, and the formula (4) will give the thickness of the abutments.

The Table IV. of M. Petit gives the thrust for those arches whose rise f is the 4th, 5th, 6th, 7th, 10th, and 16th part of the span L, and for the different values of K; these are the kinds of arches most in use. We find also in these Tables, and for each of these cases, the

value of a, and that of r, which corresponds with it. Let θ be greater or less than a, formula (4) must be used to find the thickness of the abutments; only if θ be greater than a, we must take the values of c found in Table IV.; and if θ be less than a, those in Table I. As an example, let us find the thickness of the abutments of an arch in the form of a circular arc with parallel extrados, where the angle at the centre $= 70°$ or $a = 35°$, and where the span $L = 49{\cdot}2$, and height $h = 13{\cdot}12$, we shall have $a = \frac{35}{90} \cdot \frac{3{\cdot}1416}{2} = {\cdot}6109$; $\sin. a = {\cdot}5735$ nearly, $\cos. a = {\cdot}8191$, $f = r\,(1 - \cos. a) = {\cdot}1809\,r$; $r = \frac{L}{2 \sin. a} = \frac{L}{1{\cdot}147}$ $\therefore$ $L = 1{\cdot}147\,r$; $\frac{L}{f} = \frac{1{\cdot}147\,r}{{\cdot}1809\,r} = 6{\cdot}34$.

Let us also suppose the thickness of the arch $= 3{\cdot}28$ feet, $R = r + 3{\cdot}28$; $r = \frac{L}{1{\cdot}147} = \frac{49{\cdot}2}{1{\cdot}147} = 42{\cdot}89$, and Table I. gives for $K = \frac{46{\cdot}17}{42{\cdot}89} = 1{\cdot}076$, an angle of rupture $= 49°48'$, which is greater than the angle a; hence the thrust must be found by Table IV. Here $K = 1{\cdot}076$ is between $K = 1{\cdot}07$ and $K = 1{\cdot}08$; and $\frac{L}{f} = 6{\cdot}34$ is between $\frac{L}{f} = 6$ and $\frac{L}{f} = 7$, we may find by proportion the value of $c = \frac{F'}{r} = {\cdot}04681$.

With this value and the others given above we must proceed as in the former example, and by means of equation (4) determine the thickness of the abutments for a strict equilibrium. And for the stability according

to La Hire we shall find the limit of the thickness by the formula $e = r\sqrt{3 \cdot 8\, c}$.

It still remains for us to calculate the thrust due to sliding, in order to substitute it in formula (4), in the case where it is more powerful than that due to rotation.

If the half angle at the centre, or a, be greater than 26°, the horizontal thrust due to sliding is calculated by the formula

$$F' = \cdot 15304\ (K^2 - 1)\ r^2.$$

If the half angle a is less than 26°, we must substitute its value instead of θ in the formula

$$G = \tfrac{1}{2}\, r^2\ (K^2 - 1) . \frac{\theta}{\tan.\ (\theta + 30^\circ)},$$

and we shall have the thrust due to sliding on the joint at the springings. A horizontal interline drawn in the columns indicates for all the Tables the value of K where the one thrust exceeds the other.

Arches in the form of a circular arc with horizontal extrados.

32. After finding the radii R and r as before, and consequently K, by the formula

$$F = \frac{r^2 \sin.^2 \theta}{6\,(K - \cos.\theta)} \left\{ K^2\,[6 - 3K - (3 - 2K) \cos.\theta] - \left(\frac{3\,\theta}{\sin.\theta} - \frac{1}{\cos.^2 \frac{1}{2}\theta} \right) \right\}$$

we must find the angle θ which corresponds to the maximum thrust by proceeding as in page 56. If it be less than a, or half the angle at the centre, this will be the angle of rupture, and the corresponding value of F or F' will be the value of the horizontal thrust.

If the angle θ, which answers to the maximum value

of F, is greater than a, we must put a instead of θ in the above formula. We must find the maximum thrust due to sliding by the equation

$$G = \frac{r^2 \sin.\theta}{\tan.(\theta+\phi)}\left\{K^2\left(1-\tfrac{1}{2}\cos.\theta\right)-\tfrac{1}{3}\frac{\theta}{\sin.\theta}\right\},$$

ϕ being the angle of friction of the masonry. M. Petit takes $\phi=30°$.

The greatest value of F or G must be substituted in the equation

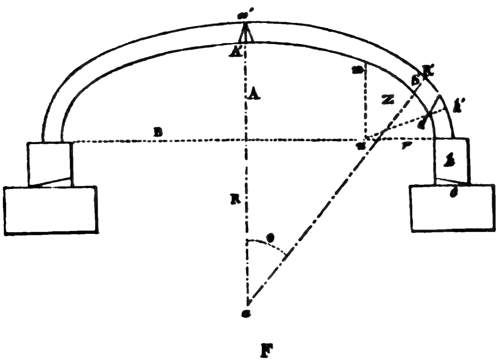
$$(R+h-r\cos.a)\,e^2+(L\,R-\tfrac{1}{2}L\,r\cos.a-r^2a)\,e+L^2\left(\tfrac{1}{4}R-\tfrac{1}{12}r\cos.a\right)-\tfrac{1}{2}L\,r^2a+\tfrac{2}{3}L\,r^2a+\tfrac{2}{3}r^3(1-\cos.a)$$
$$=2F'(R+h-r\cos.a)$$ in order to obtain e.

We need only observe that $\frac{F'}{r^2}=c$ or $F'=cr^2$, and that c is given by the Tables.

We may proceed to the formulæ of Col. Audoy for the calculation of other arches.

Formulæ relating to flat sweeps of three circular arcs with extrados parallel to the intrados.

33. Let the angle $A'aS$ (which subtends the $\frac{1}{2}$ arc at the summit)$=\theta$; let a be the thickness of the arch, or $A'a'$,



R the radius of the arc at the crown, r the radii of the arcs at the springings, A the spring, or the height of the intrados under the key-stone, B the $\frac{1}{2}$ of the span, Z the angle mud contained by the supposed joint of rupture $d\,h'$ and the vertical passing through u the centre of the small arc at the springings, h and e the height and thickness of the abutments, S′ and Z′ the surfaces A′ S R′ a', and S $d\,h'$ R′; N′ and L′ the moments of these surfaces taken with respect to the verticals A′ a and mu; M′ M″ the moments of these surfaces taken with respect to the point d; F the value of the horizontal force; we must employ the formulæ

$$S' = \left\{\frac{(R+a)^2 - R^2}{2}\right\}\theta,\quad Z' = \left\{\frac{(r+a)^2 - r^2}{2}\right\}(Z-\theta)$$

$$N' = \left\{\frac{(R+a)^3 - R^3}{3}\right\}(1-\cos.\,\theta)$$

$$L' = \left\{\frac{(r+a)^3 - r^3}{3}\right\}(\cos.\,\theta - \cos.\,Z)$$

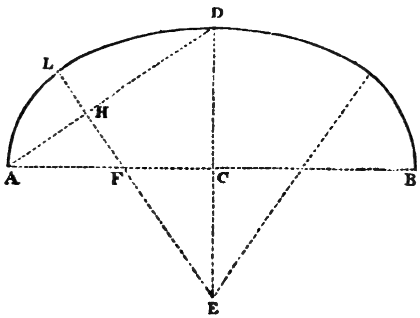
$$M' = \left\{R - r\,(1-\sin.\,Z)\right\}S' - N';\ M'' = r\sin.\,Z.\,Z' - L',$$

$$F = \frac{M'+M''}{A+a-r\cos.\,Z}.$$

By taking for Z any angle greater than θ, we may calculate all these values, and have the value of F corresponding to that angle. Similarly, we may find other values of F, by assuming different values of Z, and when we have obtained F′ the maximum value of F, we must substitute it in the equation $\frac{e^2h}{2} + e\,(S'+S'') + B\,S' + rS'' - (N'+N'') = F'(A+a+h)$ (5). S″ is the surface of B′ SR′b, which is given by the equation $S'' = \left\{\frac{(r+a)^2 - r^2}{4}\right\}(\pi - 2\,\theta)$; N″ is its moment with respect to the vertical

mu, and is given by the equation $N''=\left\{\frac{(r+a)^3-r^3}{3}\right\}$ cos. θ.

For the methods of describing these arches, see "Papers on Bridges," pages 40 to 44.



When the arcs are 60°, and angle F E C =30°, then sin. 30°= ·5 and F C=F E sin. 30°=$\frac{1}{2}$ E F=$\frac{1}{2}$ (R$-r$) or R$-r$=2 F C, but r=A C $-$ F C, hence R=A C+F C.

Consequently we have in this case the radii; R=A C +F C and r=A C$-$F C respectively.

Let us give an example in which AC=30 feet or 10 yards, C D=4$\frac{1}{2}$ feet or 1$\frac{1}{2}$ yards, and the height of the pier=12 feet or 4 yards, also C D=$\frac{2}{3}$ A C, and since F E C =30°=θ; sin. 30=·5; $\theta=\frac{30}{90}\cdot\frac{3\cdot1416}{2}=\cdot5236$; B C =$\frac{2}{3}$. 10=6$\frac{2}{3}$; and by "Papers on Bridges," (page 43,) $FC=\frac{\sqrt{3}+1}{2}(B-A)=4\cdot55342$; R = A C + F C = 14·55342; r=A C$-$F C=5·44658.

Substituting these values in the above formulæ, we have S′=12·01871; N′=47·11; Z′=9·295 (Z$-$·5236); L′=57·878 (·866$-$cos. Z); M″=5·44658 sin. Z$\Big\{$9·295 (Z$-$·5236)$\Big\}-$57·878 (·866$-$cos. Z).

$$M'=\Big\{10-5\cdot44658\ (1-\text{sin. } Z)\Big\}\ 12\cdot01871-47\cdot11,$$

$$A+a-r\ \text{cos. } Z=8\cdot16666-5\cdot44658\ \text{cos. } Z.$$

Let $Z=45°$

$$Z=\frac{45}{90}\cdot\frac{3\cdot1416}{2}=\cdot7854;\ \cos. Z=\sin. Z=\cdot7071;\ M'=53\cdot9,$$

$$M''=\cdot166,\ A+a-r\cos. Z=4\cdot3154$$

$$F=\frac{M'+M''}{A+a-r\cos. Z.}=12\cdot5286.$$

If $Z=46°$,

$Z'=2\cdot5951640$, $L'=9\cdot897138$, $M''=\cdot269902$,
$M'=54\cdot6968742$, $A+a-r\cos. Z=4\cdot38$, $F=12\cdot54$.

If $Z=47°$,

$Z'=2\cdot7578265$, $L'=10\cdot651114706$, $M''=\cdot331474387$,
$M'=55\cdot49128068545732366$,
$A+a-r\cos. Z=4\cdot452156901194$, $F=12\cdot5383$.

Here the greatest value of F found is therefore $F=12\cdot54=F'$, and corresponds to the angle of rupture $Z=46°$.

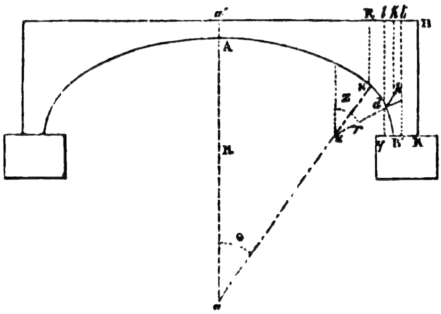
For the thickness of the piers in the case of strict equilibrium we must substitute the different terms in formula (5)
or $h=4$, $\theta=\cdot5236$, $\cos.\theta=\cdot866$, $\pi.=31416$; $F'=12\cdot54$, $a=1\cdot5$, $A=6\cdot666$, $S'=12\cdot01871$, $B=10$, $r=5\cdot44658$, $N'=47\cdot11$, $S''=9\cdot73372$, $N''=50\cdot12235$, then

$$e=-5\cdot4431-\sqrt{67\cdot92718}=2\cdot8.$$

To find the practical thickness we must increase the value of F' by $\frac{9}{10}$; before we introduce it in the formula (5), we should have substituted in that formula $12+54+\frac{9}{10}\times12\cdot54=23\cdot83$ instead of $12\cdot54$, and the resulting value would be that which must be given to the thickness of the piers, in order that they may be able to resist any accidental causes.

Formulæ relating to flat sweeps, described with three arcs of a circle with horizontal extrados.

34. Let R be the radius of the arc at the crown, r the radius of the arc at the springings, e the thickness B′K at the springings, a the thickness of the arch at the key-stone, θ the angle Aas, Z the supposed angle of rupture, S′ and Z′ the surfaces ASRa' and SdlR, N′ and L′ the moments of these surfaces with respect to the verticals Aa and mu, M′ and M″ the moments of these surfaces with respect to the vertical dq, M‴ the moment of the surface $dhh'l$ with respect to the vertical dq, F′ the maximum value of the thrust F. The formulæ are



$$S'=\frac{R\sin.\theta}{2}\left\{2(R+a)-R\cos.\theta\right\}-\frac{R^2\theta}{2};$$

$$N'=\frac{R^2\sin.^2\theta}{2}(R+a)-\frac{R^3}{3}(1-\cos.^3\theta);$$

$$Z'=r(A+a)(\sin.Z-\sin.\theta)-\frac{r^2}{2}\left\{Z+\sin.Z\cos.Z)\right.$$
$$\left.-(\theta+\sin.\theta\cos.\theta)\right\};$$

$$L'=\frac{r^2}{2}(A+a)(\sin.^2Z-\sin.^2\theta)-\frac{r^3}{3}(\cos.^3\theta-\cos.^3Z)$$

$$M'=(B-r+r\sin.Z)\,S'-N';$$

$$M''=r\sin.Z\,.\,Z'-L';\ M'''=a^2\sin.^2Z$$
$$\left\{\frac{(A+a)-r\cos.Z}{2}-\frac{a\cos.Z}{3}\right\}$$

$F = \frac{M'+M''-M'''}{A+a-r\cos. Z}$. The equation which gives the thickness e is $e^2\left(\frac{A+a}{2}\right) + P' + P'' = F(A+a)$ or e^2 $\left(\frac{A+a}{2}\right) + e(S'+S'') + BS' + rS'' - (N'+N'') = F(A+a)$.

S'' represents the surface $S B' b' R$, which is found by the equation

$$S'' = r(A+a)(1-\sin.\theta) - \frac{r^2}{2}\left(\frac{\pi-2\theta}{2} - \sin.\theta\cos.\theta\right);$$

N'' is its moment with respect to $m u$, which is

$$\frac{r^2}{2}(A+a)(1-\sin.^2\theta) - \frac{r^3\cos.^3\theta}{3};$$

P' and P'' the moments of the surfaces $A S R a'$ and $S B' b' R$ with respect to $H k$, which are found by the equations

$$P' = (e+B)S' - N';\ P'' = (e+r)S'' - N''.$$

If the thickness of the piers of an arch $= e$, the height $= h$, the equation of equilibrium is

$$e^2\frac{(A+a+h)}{2} + e(S'+S'') + B S' + r S'' - (N'+N'') = F(A+a+h).$$

Formulæ for Plate Bandes.

35. In these arches the joints cannot be perpendicular to the intrados; they are so constructed that all the joints produced meet in the same point, (see page 29). Retaining the same notation as at page 28,

$$F = \frac{3a^2 - t^2 . \tan.^2\theta}{6}, \text{ and}$$

$$c^2\left(\frac{h+t}{2}\right) + e \times t \times a + \frac{a^2 t}{2} = F(a+h).$$

It is usual to construct an equilateral triangle upon the whole breadth, which finds 0 the common centre of all the joints. In this case we have $\theta = 30°$; $\tan.\,\theta = \frac{1}{\sqrt{3}}$ and $F = \frac{9\,a^2 - t^2}{18}$.

Let $a = 3{\cdot}5$ yards, $t = {\cdot}8$ yard, and $h = 3$ yards, and $F = \frac{9 \times (3{\cdot}5)^2 - ({\cdot}8)^2}{18} = 6{\cdot}089$, and $e^2 + 1{\cdot}47\,e = 9{\cdot}6$ $\therefore$ $e = 2{\cdot}445$ for the case of strict equilibrium. To obtain the practical thickness we must proceed as in page 68.

36. In concluding this part we may remark that the investigation of many other valuable formulæ is given in the Memoir of M.M. Lamé and Clapeyron, nearly the same as we have given from M.M. Petit and Audoy.

They determine the position of the point of rupture of the spherical extradossed dome of the church of St. Isaac at Petersburg, whose radii of intrados and extrados are 32 and 34 feet respectively, which give for the thickness of the dome the $\frac{1}{32}$ part of the interior diameter, and they find that this point is 68° 18′ from the key, or, which is the same, 21° 42′ from the springings.

In the supplement they give an investigation relative to circular cylindrical arches with horizontal extrados, as follows:

Retaining the same notation as at pages 43 and 44,

$$\frac{m\,a}{k} = \frac{3\,R\,r^2 \sin.{}^2\theta - r^3 \sin.{}^2\theta \cos.\,\theta - r^3\{3\,\theta \sin.\,\theta - 2\,(1 - \cos.\,\theta)\}}{6\,(R - r\cos.\,\theta)} \quad . \; . \; (a)$$

Differentiating the second member of this equation for θ, and putting the result equal to nothing, making $\frac{R}{r} = K$

and $\frac{\theta}{\sin.\theta} = z$, we have the maximum of $\frac{m\,a}{h}$,

$$3z(1-\text{K}\cos.\theta)+2\cos.^3\theta-6\text{K}\cos.^2\theta+3(2\text{K}^2+1)\cos.\theta=3(\text{K}+2) \quad (b)$$

Calculating in the same manner as at page 46, we obtain the arc corresponding to the point of rupture for each particular value of K or $\frac{\text{R}}{r}$, and this substituted in equation (a), will give the maximum value of $\frac{m\,a}{h}$.

They also deduce the following beautiful property, viz. the point of rupture in an arch is that for which the tangent to the intrados at this point cuts the horizontal line passing through the summit of the key at the same point as the vertical passing through the centre of gravity of the mass which tends to separate itself. The point of rupture being known, $\frac{m\,a}{h}$ is then equal to the mass m divided by the tangent of the angle which the line touching the intrados at the point of rupture makes with the horizon.

They further proceed to examine if there do not exist curves which give a constant moment of stability, and find that this condition cannot be satisfied throughout the whole arch, but it may be so for a portion of the arch. For as $\frac{\text{M A}}{\text{H}}$ is constant in the same arch, they try if $\frac{m\,a}{h}$ can be so too, and find two different cases: in the first, the thickness of the arch must be $= 0$, in the second, the height of the key must $= 0$, neither of which is admissible;—thus the moment of stability cannot be constant for the whole extent of the same arch.

TABLE I.—*Semicircular arches in which the extrados is parallel to the intrados.*

Value of the ratio $K=\frac{R}{r}$	Angle of rupture.	Ratio of the thrust C to the square of the radius r of intrados.		Ratio $\sqrt{2c}$ of the limit of the thickness of the piers to the radius of intrados.	
		For the case of rotation.	For the case of sliding.	Strict equilibrium.	Equilibrium according to La Hire.
2·732	0° 00′	0·00000	0·98923	,,	,,
2·70	13 42	0·00211	0·96262	,,	,,
2·65	22 00	0·00319	0·92168	,,	,,
2·60	27 30	0·00809	0·88151	,,	,,
2·50	35 52	0·02283	0·80346	,,	,,
2·40	42 6	0·04109	0·72847	,,	,,
2·30	46 47	0·06835	0·65654	,,	,,
2·20	51 4	0·08648	0·58767	,,	,,
2·10	54 27	0·10926	0·52186	,,	,,
2·00	57 17	0·13017	0·45912	0·9582	1·3223
1·90	59 37	0·14813	0·39943	0·8938	1·2320
1·80	61 24	0·16373	0·34281	0·8280	1·1414
1·70	62 53	0·17180	0·28924	0·7606	1·0484
1·60	63 49	0·17517	0·23874	0·6910	0·9525
1·59	63 52	0·17533	0·23386	0·6839	0·9427
1·58	63 55	0·17535	0·22901	0·6768	0·9329
1·57	63 58	0·17524	0·22434	0·6698	0·9233
1·56	64 1	0·17499	0·21940	0·6624	0·9131
1·55	64 3	0·17478	0·21464	0·6552	0·9031
1·54	64 5	0·17445	0·20991	0·6479	0·8931
1·53	64 7	0·17397	0·20521	0·6406	0·8831
1·52	64 8	0·17352	0·20054	0·6333	0·8730
1·51	64 8	0·17310	0·19590	0·6259	0·8628
1·50	64 9	0·17254	0·19130	0·6185	0·8527
1·49	64 8	0·17180	0·18673	0·6111	0·8424
1·48	64 8	0·17095	0·18218	0·6036	0·8320
1·47	64 7	0·17008	0·17766	0·5961	0·8216
1·46	64 6	0·16915	0·17318	0·5885	0·8112
1·45	64 5	0·16798	0·16872	0·5809	0·8007
1·44	64 3	0·16683	0·16430	0·5776	0·7962
1·43	64 00	0·16568	0·15991	0·5756	0·7934
1·42	63 56	0·16448	0·15555	0·5735	0·7906
1·41	63 52	0·16317	0·15122	0·5713	0·7874
1·40	63 48	0·16167	0·14691	0·5686	0·7838
1·39	63 43	0·16014	0·14264	0·5659	0·7801

Value of the ratio $K=\frac{R}{r}$	Angle of rupture.	Ratio of the thrust C to the square of the radius r of intrados.		Ratio $\sqrt{2}\,e$ of the limit of the thickness of the piers to the radius of intrados.	
		For the case of rotation.	For the case of sliding.	Strict equilibrium.	Equilibrium according to La Hire.
1·38	63° 38′	0·15845	0·13841	0·5629	0·7760
1·37	63 32	0·15672	0·13420	0·5598	0·7717
1·36	63 26	0·15482	0·13002	0·5564	0·7670
1·35	63 19	0·15287	0·12587	0·5529	0·7622
1·34	63 10	0·15096	0·12176	0·5495	0·7574
1·33	63 10	0·14896	0·11767	0·5458	0·7524
1·32	62 50	0·14678	0·11362	0·5418	0·7468
1·31	62 33	0·14510	0·10959	0·5387	0·7425
1·30	62 14	0·14330	0·10559	0·5353	0·7379
1·29	62 9	0·14013	0·10163	0·5294	0·7297
1·28	62 3	0·13691	0·09770	0·5233	0·7213
1·27	61 47	0·13430	0·09379	0·5183	0·7144
1·26	61 30	0·13157	0·08992	0·5130	0·7071
1·25	61 15	0·12847	0·08608	0·5069	0·6987
1·24	61 1	0·12516	0·08227	0·5003	0·6896
1·23	60 40	0·12201	0·07849	0·4940	0·6809
1·22	60 19	0·11887	0·07474	0·4876	0·6721
1·21	60 00	0·11516	0·07102	0·4799	0·6615
1·20	59 41	0·11140	0·06733	0·4720	0·6504
1·19	59 10	0·10791	0·06368	0·4646	0·6404
1·18	58 40	0·10417	0·06005	0·4564	0·6292
1·17	58 9	0·10021	0·05646	0·4472	0·6171
1·16	57 40	0·09593	0·05289	0·4380	0·6038
1·15	57 1	0·09176	0·04935	0·4284	0·5905
1·14	56 23	0·08729	0·04585	0·4178	0·5759
1·13	55 45	0·08254	0·04237	0·4063	0·5601
1·12	54 48	0·07789	0·03984	0·3947	0·5444
1·11	54 10	0·07273	0·03552	0·3814	0·5259
1·10	53 15	0·06754	0·03213	0·3675	0·5066
1·09	52 14	0·06177	0·02879	,,	,,
1·08	51 7	0·05649	0·02546	,,	,,
1·07	49 48	0·05065	0·02217	,,	,,
1·06	48 18	0·04455	0·01891	,,	,,
1·05	46 32	0·03813	0·01568	,,	,,
1·04	44 4	0·03139	0·01249	,,	,,
1·03	41 4	0·02459	0·00932	,,	,,
1·02	38 12	0·01691	0·00618	,,	,,
1·01	32 36	0·00889	0·00308	,,	,,
1·00	0 00	0·00000	0·00000	,,	,,

TABLE II.—*Semicircular arches with parallel extrados.—Table of the thickness of the piers.*

Value of the ratio $K=\frac{R}{r}$	Ratio $\frac{e}{r}$ of the thickness of the piers to the radius of intrados as a function of the ratio $\frac{r}{h}$ of radius to the height of the piers. (For strict equilibrium.)
2·00	$-2{\cdot}3562\frac{r}{h}+\sqrt{(5{\cdot}5517\frac{r^2}{h^2}+1{\cdot}7907\frac{r}{h}+0{\cdot}9182)}$
1·90	$-2{\cdot}0449\frac{r}{h}+\sqrt{(4{\cdot}2021\frac{r^2}{h^2}+1{\cdot}3240\frac{r}{h}+0{\cdot}7988)}$
1·80	$-1{\cdot}7593\frac{r}{h}+\sqrt{(3{\cdot}0951\frac{r^2}{h^2}+0{\cdot}9368\frac{r}{h}+0{\cdot}6856)}$
1·70	$-1{\cdot}4844\frac{r}{h}+\sqrt{(2{\cdot}2034\frac{r^2}{h^2}+0{\cdot}6933\frac{r}{h}+0{\cdot}5785)}$
1·60	$-1{\cdot}2252\frac{r}{h}+\sqrt{(1{\cdot}5012\frac{r^2}{h^2}+0{\cdot}3775\frac{r}{h}+0{\cdot}4775)}$
1·59	$-1{\cdot}2001\frac{r}{h}+\sqrt{(1{\cdot}4404\frac{r^2}{h^2}+0{\cdot}3566\frac{r}{h}+0{\cdot}4677)}$
1·58	$-1{\cdot}1752\frac{r}{h}+\sqrt{(1{\cdot}3812\frac{r^2}{h^2}+0{\cdot}3361\frac{r}{h}+0{\cdot}4580)}$
1·57	$-1{\cdot}1513\frac{r}{h}+\sqrt{(1{\cdot}3255\frac{r^2}{h^2}+0{\cdot}3151\frac{r}{h}+0{\cdot}4487)}$
1·56	$-1{\cdot}1261\frac{r}{h}+\sqrt{(1{\cdot}2677\frac{r^2}{h^2}+0{\cdot}2966\frac{r}{h}+0{\cdot}4388)}$
1·55	$-1{\cdot}1015\frac{r}{h}+\sqrt{(1{\cdot}2133\frac{r^2}{h^2}+0{\cdot}2783\frac{r}{h}+0{\cdot}4293)}$
1·54	$-1{\cdot}0772\frac{r}{h}+\sqrt{(1{\cdot}1605\frac{r^2}{h^2}+0{\cdot}2603\frac{r}{h}+0{\cdot}4198)}$
1·53	$-1{\cdot}0531\frac{r}{h}+\sqrt{(1{\cdot}1091\frac{r^2}{h^2}+0{\cdot}2428\frac{r}{h}+0{\cdot}4104)}$
1·52	$-1{\cdot}0292\frac{r}{h}+\sqrt{(1{\cdot}0592\frac{r^2}{h^2}+0{\cdot}2224\frac{r}{h}+0{\cdot}4011)}$
1·51	$-1{\cdot}0073\frac{r}{h}+\sqrt{(1{\cdot}0146\frac{r^2}{h^2}+0{\cdot}2056\frac{r}{h}+0{\cdot}3918)}$
1·50	$-0{\cdot}9817\frac{r}{h}+\sqrt{(0{\cdot}9638\frac{r^2}{h^2}+0{\cdot}1937\frac{r}{h}+0{\cdot}3826)}$
1·49	$-0{\cdot}9583\frac{r}{h}+\sqrt{(0{\cdot}9184\frac{r^2}{h^2}+0{\cdot}1684\frac{r}{h}+0{\cdot}3735)}$
1·48	$-0{\cdot}9349\frac{r}{h}+\sqrt{(0{\cdot}8741\frac{r^2}{h^2}+0{\cdot}1659\frac{r}{h}+0{\cdot}3644)}$

Value of the ratio $K = \frac{R}{r}$	Ratio $\frac{e}{r}$ of the thickness of the piers to the radius of intrados as a function of the ratio $\frac{r}{h}$ of radius to the height of the piers. (For strict equilibrium.)
1·47	$-0{\cdot}9125\frac{r}{h} + \sqrt{(0{\cdot}8328\frac{r^2}{h^2} + 0{\cdot}1482\frac{r}{h} + 0{\cdot}3553)}$
1·46	$-0{\cdot}8887\frac{r}{h} + \sqrt{(0{\cdot}7899\frac{r^2}{h^2} + 0{\cdot}1362\frac{r}{h} + 0{\cdot}3464)}$
1·45	$-0{\cdot}8659\frac{r}{h} + \sqrt{(0{\cdot}7498\frac{r^2}{h^2} + 0{\cdot}1232\frac{r}{h} + 0{\cdot}3374)}$
1·44	$-0{\cdot}8432\frac{r}{h} + \sqrt{(0{\cdot}7110\frac{r^2}{h^2} + 0{\cdot}1181\frac{r}{h} + 0{\cdot}3337)}$
1·43	$-0{\cdot}8206\frac{r}{h} + \sqrt{(0{\cdot}6735\frac{r^2}{h^2} + 0{\cdot}1153\frac{r}{h} + 0{\cdot}3314)}$
1·42	$-0{\cdot}7983\frac{r}{h} + \sqrt{(0{\cdot}6372\frac{r^2}{h^2} + 0{\cdot}1143\frac{r}{h} + 0{\cdot}3290)}$
1·41	$-0{\cdot}7760\frac{r}{h} + \sqrt{(0{\cdot}6023\frac{r^2}{h^2} + 0{\cdot}1102\frac{r}{h} + 0{\cdot}3263)}$
1·40	$-0{\cdot}7540\frac{r}{h} + \sqrt{(0{\cdot}5685\frac{r^2}{h^2} + 0{\cdot}1074\frac{r}{h} + 0{\cdot}3233)}$
1·39	$-0{\cdot}7321\frac{r}{h} + \sqrt{(0{\cdot}5359\frac{r^2}{h^2} + 0{\cdot}1048\frac{r}{h} + 0{\cdot}3203)}$
1·38	$-0{\cdot}7103\frac{r}{h} + \sqrt{(0{\cdot}5045\frac{r^2}{h^2} + 0{\cdot}1021\frac{r}{h} + 0{\cdot}3169)}$
1·37	$-0{\cdot}6887\frac{r}{h} + \sqrt{(0{\cdot}4743\frac{r^2}{h^2} + 0{\cdot}0995\frac{r}{h} + 0{\cdot}3134)}$
1·36	$-0{\cdot}6673\frac{r}{h} + \sqrt{(0{\cdot}4452\frac{r^2}{h^2} + 0{\cdot}0969\frac{r}{h} + 0{\cdot}3096)}$
1·35	$-0{\cdot}6460\frac{r}{h} + \sqrt{(0{\cdot}4173\frac{r^2}{h^2} + 0{\cdot}0944\frac{r}{h} + 0{\cdot}3057)}$
1·34	$-0{\cdot}6249\frac{r}{h} + \sqrt{(0{\cdot}3904\frac{r^2}{h^2} + 0{\cdot}0926\frac{r}{h} + 0{\cdot}3019)}$
1·33	$-0{\cdot}6050\frac{r}{h} + \sqrt{(0{\cdot}3660\frac{r^2}{h^2} + 0{\cdot}0903\frac{r}{h} + 0{\cdot}2979)}$
1·32	$-0{\cdot}5831\frac{r}{h} + \sqrt{(0{\cdot}3400\frac{r^2}{h^2} + 0{\cdot}0880\frac{r}{h} + 0{\cdot}2936)}$
1·31	$-0{\cdot}5624\frac{r}{h} + \sqrt{(0{\cdot}3163\frac{r^2}{h^2} + 0{\cdot}0875\frac{r}{h} + 0{\cdot}2902)}$
1·30	$-0{\cdot}5419\frac{r}{h} + \sqrt{(0{\cdot}2937\frac{r^2}{h^2} + 0{\cdot}0867\frac{r}{h} + 0{\cdot}2866)}$
1·29	$-0{\cdot}5216\frac{r}{h} + \sqrt{(0{\cdot}2720\frac{r^2}{h^2} + 0{\cdot}0828\frac{r}{h} + 0{\cdot}2803)}$

Value of the ratio $K = \frac{R}{r}$	Ratio $\frac{e}{r}$ of the thickness of the piers to the radius of intrados as a function of the ratio $\frac{r}{h}$ of radius to the height of the piers. (For strict equilibrium.)
1·28	$-0{\cdot}5014\frac{r}{h}+\sqrt{(0{\cdot}2520\frac{r^2}{h^2}+0{\cdot}0801\frac{r}{h}+0{\cdot}2738)}$
1·27	$-0{\cdot}4926\frac{r}{h}+\sqrt{(0{\cdot}2426\frac{r^2}{h^2}+0{\cdot}0778\frac{r}{h}+0{\cdot}2686)}$
1·26	$-0{\cdot}4615\frac{r}{h}+\sqrt{(0{\cdot}2130\frac{r^2}{h^2}+0{\cdot}0755\frac{r}{h}+0{\cdot}2631)}$
1·25	$-0{\cdot}4418\frac{r}{h}+\sqrt{(0{\cdot}1952\frac{r^2}{h^2}+0{\cdot}0730\frac{r}{h}+0{\cdot}2569)}$
1·24	$-0{\cdot}4222\frac{r}{h}+\sqrt{(0{\cdot}1783\frac{r^2}{h^2}+0{\cdot}0713\frac{r}{h}+0{\cdot}2503)}$
1·23	$-0{\cdot}4028\frac{r}{h}+\sqrt{(0{\cdot}1623\frac{r^2}{h^2}+0{\cdot}0684\frac{r}{h}+0{\cdot}2440)}$
1·22	$-0{\cdot}3836\frac{r}{h}+\sqrt{(0{\cdot}1471\frac{r^2}{h^2}+0{\cdot}0674\frac{r}{h}+0{\cdot}2377)}$
1·21	$-0{\cdot}3645\frac{r}{h}+\sqrt{(0{\cdot}1329\frac{r^2}{h^2}+0{\cdot}0641\frac{r}{h}+0{\cdot}2303)}$
1·20	$-0{\cdot}3456\frac{r}{h}+\sqrt{(0{\cdot}1194\frac{r^2}{h^2}+0{\cdot}0614\frac{r}{h}+0{\cdot}2228)}$
1·19	$-0{\cdot}3268\frac{r}{h}+\sqrt{(0{\cdot}1068\frac{r^2}{h^2}+0{\cdot}0600\frac{r}{h}+0{\cdot}2158)}$
1·18	$-0{\cdot}3082\frac{r}{h}+\sqrt{(0{\cdot}0950\frac{r^2}{h^2}+0{\cdot}0581\frac{r}{h}+0{\cdot}2083)}$
1·17	$-0{\cdot}2897\frac{r}{h}+\sqrt{(0{\cdot}0840\frac{r^2}{h^2}+0{\cdot}0561\frac{r}{h}+0{\cdot}2004)}$
1·16	$-0{\cdot}2714\frac{r}{h}+\sqrt{(0{\cdot}0734\frac{r^2}{h^2}+0{\cdot}0559\frac{r}{h}+0{\cdot}1919)}$
1·15	$-0{\cdot}2533\frac{r}{h}+\sqrt{(0{\cdot}0642\frac{r^2}{h^2}+0{\cdot}0536\frac{r}{h}+0{\cdot}1835)}$
1·14	$-0{\cdot}2353\frac{r}{h}+\sqrt{(0{\cdot}0554\frac{r^2}{h^2}+0{\cdot}0513\frac{r}{h}+0{\cdot}1745)}$
1·13	$-0{\cdot}2175\frac{r}{h}+\sqrt{(0{\cdot}0473\frac{r^2}{h^2}+0{\cdot}0490\frac{r}{h}+0{\cdot}1651)}$
1·12	$-0{\cdot}1998\frac{r}{h}+\sqrt{(0{\cdot}0399\frac{r^2}{h^2}+0{\cdot}0467\frac{r}{h}+0{\cdot}1557)}$
1·11	$-0{\cdot}1823\frac{r}{h}+\sqrt{(0{\cdot}0332\frac{r^2}{h^2}+0{\cdot}0426\frac{r}{h}+0{\cdot}1455)}$
1·10	$-0{\cdot}1649\frac{r}{h}+\sqrt{(0{\cdot}0272\frac{r^2}{h^2}+0{\cdot}0394\frac{r}{h}+0{\cdot}1351)}$

Table III.—*Semicircular arches in which the extrados is horizontal.—Table of the angles of rupture, of the thrusts, and the limit of the thickness of the piers.*

Value of the ratio $K=\frac{R}{r}$	Angle of rupture.	Ratio C of the thrust to the square of the radius r of intrados.		Ratio of the limit of the thickness of the piers to the radius r of intrados.	
		For rotation.	For sliding.	Strict equilibrium.	Stability according to La Hire.
2·00	36°	0·05486	0·50358	1·0036	1·3834
1·90	39	0·07101	0·43966	0·9377	1·2925
1·80	44	0·08850	0·37901	0·8706	1·2001
1·70	48	0·10631	0·32164	0·8020	1·1055
1·60	52	0·12300	0·26755	0·7315	1·0082
1·59	52	0·12453	0·26232	0·7243	0·9984
1·58	53	0·12602	0·25712	0·7171	0·9885
1·57	53	0·12747	0·25196	0·7099	0·9784
1·56	54	0·12837	0·24683	0·7026	0·9684
1·55	54	0·13027	0·24173	0·6953	0·9584
1·54	55	0·13153	0·23667	0·6880	0·9483
1·53	55	0·13289	0·23163	0·6806	0·9381
1·52	55	0·13414	0·22664	0·6732	0·9280
1·51	55	0·13531	0·22167	0·6658	0·9177
1·50	56	0·13648	0·21673	0·6583	0·9075
1·49	56	0·13756	0·21183	0·6509	0·8972
1·48	56	0·13856	0·20696	0·6433	0·8868
1·47	57	0·13952	0·20213	0·6358	0·8764
1·46	57	0·14041	0·19733	0·6282	0·8659
1·45	57	0·14122	0·19256	0·6206	0·8554
1·44	58	0·14195	0·18782	0·6129	0·8448
1·43	58	0·14268	0·18312	0·6052	0·8341
1·42	58	0·14311	0·17845	0·5974	0·8234
1·41	59	0·14376	0·17381	0·5896	0·8126
1·40	59	0·14421	0·16920	0·5817	0·8018
1·39	59	0·14456	0·16463	0·5738	0·7909
1·38	59	0·14481	0·16009	0·5658	0·7799
1·37	60	0·14498	0·15558	0·5578	0·7689
1·36	60	0·14506	0·15111	0·5497	0·7577

Value of the ratio $K = \frac{R}{r}$	Angle of rupture.	Ratio C of the thrust to the square of the radius r of intrados.		Ratio of the limit of the thickness of the piers to the radius r of intrados.	
		For rotation.	For sliding.	Strict equilibrium.	Stability according to La Hire.
1·35	60°	0·14504	0·14666	0·5416	0·7465
1·34	60	0·14491	0·14225	0·5383	0·7420
1·33	61	0·14467	,,	0·5379	0·7414
1·32	61	0·14460	,,	0·5377	0·7412
1·31	61	0·14390	,,	0·5358	0·7394
1·30	61	0·14322	0·12495	0·5354	0·7379
1·29	61	0·14264	,,	0·5341	0·7362
1·28	62	0·14186	,,	0·5326	0·7342
1·27	62	0·14101	,,	0·5310	0·7320
1·26	62	0·13988	,,	0·5289	0·7290
1·25	62	0·13872	0·10405	0·5267	0·7260
1·24	62	0·13737	,,	0·5235	0·7225
1·23	63	0·13593	,,	0·5214	0·7187
1·22	63	0·13437	,,	0·5184	0·7145
1·21	63	0·13263	,,	0·5150	0·7099
1·20	63	0·13073	0·08397	0·5113	0·7048
1·19	63	0·12870	,,	0·5073	0·6993
1·18	63	0·12650	,,	0·5030	0·6933
1·17	64	0·12415	,,	0·4983	0·6868
1·16	64	0·12182	,,	0·4936	0·6803
1·15	64	0·11895	0·06471	0·4877	0·6723
1·14	64	0·11608	,,	0·4818	0·6641
1·13	64	0·11303	,,	0·4755	0·6553
1·12	64	0·10979	,,	0·4886	0·6459
1·11	65	0·10641	,,	0·4613	0·6358
1·10	65	0·10279	0·04627	0·4535	0·6249
1·09	66	0·098992	,,	0·4449	0·6133
1·08	66	0·094967	,,	0 4358	0·6007
1·07	67	0·091189	,,	0·4270	0·5886
1·06	68	0·086376	,,	0·4156	0·5729
1·05	69	0·081755	0·02865	0·4044	0·5573
1·04	70	0·076857	,,	,,	,,
1·03	71	0·071853	,,	,,	,,
1·02	73	0·066469	,,	,,	,,
1·01	74	0·061324	,,	,,	,,
1·00	75	0·055472	0·01185	,,	,,

TABLE IV.—*Arches in the form of an arc of a circle with parallel extrados. Tables of thrusts in various systems.*

Value of the ratio $K = \frac{R}{r}$	RATIO OF THE THRUST TO THE SQUARE OF THE RADIUS OF EXTRADOS.					
	System where $L = 4f$ $r = \frac{5}{2}f$ $a = 53°7'30''$	System where $L = 5f$ $r = \frac{29}{8}f$ $a = 43°36'10''$	System where $L = 6f$ $r = 5f$ $a = 36°52'10''$	System where $L = 7f$ $r = \frac{53}{8}f$ $a = 31°53'26''$	System where $L = 8f$ $r = \frac{17}{2}f$ $a = 28°4'20''$	System where $L = 10f$ $r = 13f$ $a = 22°37'10''$
1·40	0·15445	0·14691	0·14691	0·14691	0·14691	0·14478
1·35	0·14717	0·13030	0·12587	0·12587	0·12587	0·12405
1·34	0·14543	0·12987	0·12171	0·12171	0·12171	0·11999
1·33	0·14364	0·12781	0·11767	0·11767	0·11767	0·11596
1·32	0·14173	0·12634	0·11362	0·11362	0·11362	0·11196
1·31	0·13975	0·12486	0·10959	0·10959	0·10959	0·10800
1·30	0·13764	0·12331	0·10682	0·10559	0·10559	0·10406
1·29	0·13543	0·12164	0·10563	0·10163	0·10163	0·10016
1·28	0·13311	0·11988	0·10437	0·09770	0·09770	0·09628
1·27	0·13068	0·11803	0·10304	0·09379	0·09379	0·09244
1·26	0·12815	0·11609	0·10160	0·08992	0·08992	0·08862
1·25	0·12547	0·11402	0·10009	0·08668	0·08608	0·08483
1·24	0·12270	0·11251	0·09850	0·08549	0·08227	0·08108
1·23	0·12031	0·10958	0·09679	0·08423	0·07849	0·07735
1·22	0·11675	0·10725	0·09499	0·08291	0·07474	0·07366
1·21	0·11354	0·10460	0·09305	0·08148	0·07102	0·06999
1·20	0·11023	0·10196	0·09102	0·07999	0·06981	0·06636
1·19	0·10676	0·09915	0·08885	0·07834	0·06859	0·06275
1·18	0·10313	0·09617	0·08653	0·07651	0·06727	0·05918
1·17	0·09934	0·09303	0·08408	0·07468	0·06583	0·05212
1·16	0·09537	0·08975	0·08144	0·07264	0·06420	0·05004
1·15	0·09123	0·08634	0·07866	0·07050	0·06259	0·04904
1·14	0·08690	0·08257	0·07568	0·06812	0·06077	0·04803
1·13	0·08238	0·07869	0·07251	0·06558	0·05890	0·04671
1·12	0·07764	0·07459	0·06911	0·06297	0·05659	0·04451
1·11	0·07269	0·07042	0·06548	0·06026	0·05421	0·04384
1·10	0·06737	0·06563	0·06158	0·05666	0·05160	0·04214
1·09	0·06211	0·06077	0·05739	0·05345	0·04871	0·04023
1·08	0·05636	0·05652	0·05288	0·04934	0·04552	0·03806
1·07	0·05052	0·05011	0·04804	0·04426	0·04200	0·03560
1·06	0·04431	0·04428	0·04280	0·04058	0·03861	0·03276
1·05	0·03776	0·03804	0·03709	0·03550	0·03357	0·02944
1·04	0·03096	0·03144	0·03095	0·02992	0·02862	0·02561
1·03	0·02378	0·02437	0·02424	0·02369	0·02293	0·02131
1·02	0·01625	0·01681	0·01690	0·01673	0·01640	0·01546
1·01	0·00834	0·00871	0·00886	0·00889	0·00885	0·00862

PAPERS ON BRIDGES.

GAUTHEY.

I. ON THE GENERAL PRINCIPLES THAT OUGHT TO REGULATE THE CONSTRUCTION OF BRIDGES, AND DETERMINE THE DIMENSIONS OF THEIR SEVERAL PARTS.

The principal object to be observed in forming the plan of a bridge, is to give a suitable and convenient aperture to the arches, so as to afford a free vent to the waters of sudden floods or inundations, and to secure the solidity and duration of the edifice by a skilful construction.

The solidity of a bridge depends almost entirely on the manner in which its foundations are laid. When these are once properly arranged, the upper part may be erected either with simplicity or elegance, without impairing in any degree the durability of the structure. Experience has proved that many bridges either decay, or are swept away by sudden floods, by reason of the defective mode of fixing their foundations, while very few suffer from an unskilful construction of the piles or arches. This latter defect, however, is easy of cor-

rection, nor is it difficult to prevent the consequences that might be expected from it.

It is to be observed, that almost all the bridges erected before the eighteenth century are built with a view to economy, both with respect to the style of building, and the degree of breadth allotted to them, since the most considerable scarcely allow room for two carriages to pass abreast. The bridges of Paris are indeed, for the most part, very wide; but this extent has been given to them solely with a view to erect two rows of small houses on their sides—a circumstance that must have naturally produced a very narrow thoroughfare.

In most of the old bridges in France, the arches are not large: and, with the exception of the projections and angles of the walls, which are of free-stone, the rest of the building consists of a rough kind of stone, the cost of which is much less; yet these bridges are, however, durable and lasting. In the last century more attention was paid to elegance, and to the display of the taste and ingenuity of the architect. Even at a distance from cities or large towns, spacious bridges were erected, and raised on very long and deep sunk arches, the bold and difficult construction of which required free-stone of great size and extraordinary price. The consequence has been that these beautiful bridges reflected honour on France, and inspired foreigners with a high idea of the perfection to which the art had been carried; but the small number of the grand bridges that arose from this system has completely absorbed the resources that the government devoted to this department, and obliged it to neglect

bridges that were on roads of greater importance, and vicinities of superior population, which, however, would have been of immense utility to the general interests of commerce.

It is an indispensable consideration in these matters, to make a distinction between the different species of bridges with respect to their size, and the mode of their construction. Such as are built on roads of the second or third class, and in towns of no great extent, ought not to be formed on the same plan as the bridges over roads more frequented by foreigners, or situated at the entry, or in the interior of extensive cities. In the former class, solidity and durability ought to be kept in view; in the latter, something more is required with respect to elegance and skill.

By the adoption of these principles, the real interests of the government will be secured, and its resources will not be squandered on useless projects.

In the projection of a bridge five principal points are necessary to be considered: 1st, the choice of its position or locality; 2nd, the vent, or egress that must be allowed to the river; 3rd, the form of the arches; 4th, the size of the arches; 5th, the breadth of the bridge. Certain rules arising from the local circumstances determine the nature of these several considerations; and in the concocting of the plan nothing should ever be left to chance or caprice.

When these fundamental positions are thus laid down and determined, nothing remains but to fix the particular dimensions of each part, the solidity of the arches, and

that of the piles and buttresses; and to choose the mode of foundation best to be observed, and the nature of the materials intended for the construction of the bridge. It will, however, often happen that these latter considerations will operate on the previous determinations; and it will be necessary, in order to come to a definitive resolution, to combine together the different circumstances of locality, and have them all present to the mind at once.

An exact knowledge of the ground or locality is indispensably necessary in the formation of the plan. It will be requisite to have a sketch of the course of the river, sufficiently extensive to furnish an idea of its general nature, the changes that its bed may have experienced, or such as may subsequently occur. In cases where the position of the bridge is not previously adopted, the plan should be furnished with certain data that would lead to a selection of this important point. It is equally necessary to ascertain the level of the stream, in order to discover its slope; and this level ought to be taken during the different seasons of the year, that a due judgment may be formed respecting the variations that floods might occasion, either in the slope or declivity, or in the mode of its distribution.

In addition to the various sketches that may be drawn up descriptive of the length of the river's course, others should be taken across its bed, so as to determine its breadth, its form, and the depth of its waters at the different periods of the year; and it is especially essential to establish two fixed points, the one relative to the

lowest, and the other to the highest, elevations of the water which have been observed. These elucidations will be useful to calculate the different heights of the foundations of the various parts of the bridge, and those of the arches, as well as the steps that must be formed at the landing-places. The measure of the velocity of the water should also be annexed to these documents, and this is particularly important at the period of sudden floods.

The plan and sections of the course of the stream, and the measure of its velocity, will serve to determine the position of the bridge, and the principal dimensions of its parts. But in order to ascertain the best method of forming its foundation, the nature of the soil on which its waters flow must be investigated, which knowledge will be derived from accurate soundings taken at different parts, and at proper depths, that will lead to an accurate conclusion on this point. When we come to treat of the Foundation of Bridges, the proper mode of proceeding with respect to this operation, as well as other necessary points of information on the same subject, will be given.

It will likewise be essentially requisite to procure exact authority on the nature and value of the materials to be employed, on the possibility of collecting a sufficient number of workmen during the different seasons of the year, and on the degree of talent and skill that they may possess. The architect ought carefully to collect all these materials of information, since they constitute the primary elements of the plan which he means to pursue.

ON THE CONSTRUCTION OF BRIDGES.

II. ON THE POSITION OF BRIDGES.

THE spot on which a bridge is to be built is not always at the disposal of the constructor. In the country it is usually determined by the direction of the roads, and in the interior of cities by the position of the streets, or their immediate vicinities. Nothing then is left to the constructor but to overcome the obstacles which nature opposes to him in the quarter which is consigned to his care.

It must be acknowledged, however, that the choice of this position is sometimes arbitrary, and even in cities, where it is intended to open new streets, especially when the old ones are too narrow and intricate. This method has been adopted at Mantes, Orleans, Moulins, &c. In such a case it will be proper to erect the bridge on the most solid ground, on one not apt to shrink, or be wasted by the force of the current. Rock being always the best basis for foundations, it may be allowable sometimes to deviate in search of it when it does not lie too deep, and to lengthen the road, rather than expose the solidity and duration of the work by constructing it on

an unsafe soil. But it is clear that it is impossible to lay down general rules on this point, since the matter is generally decided by the particular circumstances of the case; and we need only point out the principal motives by which the position of the bridge ought to be regulated and determined.

It will be essentially necessary to place the axis of the bridge perpendicular to the level of the water, so that the direction of the current shall be parallel to the lateral faces of the piles. When this is not possible, the faces of the piles are inclined relatively to the axis of the bridge, which then takes the name of oblique or sloping bridge. A construction of this kind must be adopted when the road makes an angle different from a right one with the course of the river.

This species of bridge is generally avoided, principally when it has many arches, on account of the difficulty of this construction; but this difficulty, which is dependent on mere matters of detail, is far from deserving the importance usually assigned to it. No reluctance should ever be felt to build a bridge of this stamp, where any inconvenience or irregularity can be removed, either in the direction of the road, or the ordinary current of the stream.

ON THE DISCHARGE OF THE WATERS FROM BRIDGES.

This subject is one of great importance in the construction of bridges, as it operates on their durability in proportion as its due solution is either more or less

exact. This problem, however, is unfortunately too like all those which are connected with the physico-mathematical sciences, as we are obliged, in order to resolve it, to have recourse to defective data, which always lead to uncertainty in the results obtained from them, until numerous and authenticated experiments shall have removed every obscurity on the subject.

The discharge or vent that ought to be allotted to a river, is less difficult to settle when there are bridges in its vicinity, built on the same stream. In that case it will be only necessary to measure, during its highest elevation, the section of the river at the passages of those bridges, and to observe the velocity of the water, and the fall that usually arises from the upper part of the stream.

By means of conclusions drawn from these data, we may be enabled to fix the vent or discharge in a manner sufficiently exact. But if no bridge exists either above or below the intended construction, we must only refer, in order to solve this problem, to the rules which we shall lay down, and which may be usefully adopted in all cases, even though they should present nothing more than a medium of verification. The question of the discharges naturally involves two other points of inquiry—1st, to determine, after a knowledge of the bed of the river, what quantity of water the bridge should let pass; 2nd, this quantity of water being ascertained, to fix the surface or extent of the necessary discharge.

On the manner of calculating the quantity of water to which the bridge should afford a free passage.

It is evident that the volume of water discharged by any river is not the same in all the seasons of the year. Not only is it less considerable in summer than in winter, but all rivers are subject to occasional increases produced by temporary and profuse showers of rain, or by the melting of snow or ice. The discharge of a river ought therefore to be extensive enough to allow not only a passage to the mean quantity of water in the bed of the river, but also to afford a vent to the surplus quantity of water that arises from floods or inundations; and it is particularly, by paying attention to this latter circumstance, that the proper dimensions of the arches can be ascertained.

This quantity of water seems at first to preserve a regular proportion to the surface of the ground over which it runs, and approaches the point where the intended bridge is to be erected. This supposition, however, will lead to very serious errors. In fact, it has been observed that the quantity of water that falls during the same year is very different in different places; and moreover, the nature and the inclinations of the ground that receives it, has much influence on the manner in which it discharges itself with more or less velocity, or penetrates the earth to a greater or lesser depth. Besides, it is easy to see that it is not so much the quantity of water that falls during the whole year, as that which proceeds from heavy rains, or the melting of snow, that

ought to be taken into calculation, as these produce frequently an extraordinary increase. However, if in many respects this consideration does not merit much attention, it ought not to be entirely neglected; it may afford room for many useful speculations, that may be applied to places in the vicinity of each other, and where the inclination and nature of the ground may be nearly similar.

There is another circumstance well deserving of attention, which is, that of the time in which the surplus water arising from a flood takes to discharge itself along the bed of the river, or the velocity with which this discharge is made. It is well known that this degree of velocity depends, in a great measure, on the slope of the river; and as this slope diminishes regularly in proportion as it recedes from its source, it follows that the same mass of water which flows rapidly in the vicinity of the mountains where the river takes its rise, and where it is still a torrent, will proceed with greater slowness during the rest of its course, as it advances towards the sea, or to the river into which it is discharged. Thus, admitting that it does not receive any considerable afflux, and that its bed is generally level, if two bridges are built on this same river, it will be necessary to give to the bridge that shall be nearer to the fountain-head a wider extent of discharge than to the other; since they must both afford a vent to the same increase of stream, which will take two days to clear off in the former case, and eight days in the latter.

It is laid down as a rule, that in order to calculate the

quantity of water that flows in a river, it will be necessary to multiply the surface of the section by the mean velocity of the current. The first of these elements of calculation is always easy to be ascertained with sufficient precision, but that is not the case with the second. We are obliged to deduce it by means of an approximation to the velocity of some of the particles of the water of which the river is composed, and more particularly of the velocity observed in the surface, as well as the middle of the stream.

Amidst the numerous experiments made by Dubuat on the motion of fluids, we find some which have for their object to determine the proportion of the velocity on the surface with that at the bottom of the current, and its mean velocity. From these experiments he has deduced a formula with a sufficient degree of approximation; for putting V for the velocity at the surface, and U for the mean velocity, he finds this equation,

$$U = (V^{\frac{1}{2}} - \tfrac{1}{2}w)^2 + \tfrac{1}{4}w,$$

this last letter being constantly $= 0^{m}\cdot 02707$; and to express the proportion between the velocity on the surface and that at the bottom, calling the latter W, he finds

$$W = (V^{\frac{1}{2}} - w^{\frac{1}{2}})^2.$$

These two formulæ are used by him to calculate the table that gives the estimate of the velocity at the bottom, and that of the mean velocity, proportionally to the velocity on the surface, from

$$V = 0^{m}\cdot 027 \text{ to } V = 2^{m}\cdot 707.$$

M. de Prony took up the subject after him,[1] and observed that the formulæ of Dubuat were in several cases at variance with the results of experience. For supposing in the second $V = 0$, we have $W = w$, from which it follows that the velocity at the surface being nothing, the velocity of the under stream would be equal to $0^{m.}027$, a conclusion that cannot be admitted. If in the first formula we make $V = 0$, a finite value will be found for U, which also does not agree with the phenomena which Nature regularly presents. It necessarily follows that the relation established between U and V, must give at the same time $U = 0$ and $V = 0$, or $U = \infty$ and $V = \infty$.

M. de Prony having laboured to find out a formula in accordance with both these conditions, makes choice of an equation of this kind,

$$U = \frac{V\,(V + a)}{V + b};$$

and the values of the invariable quantities a and b being determined by seventeen experiments of Dubuat, made on the values of V, which varied from $V = 0^{m.}15$ to $V = 1^{m.}30$, by a method of correcting the anomalies explained in the work which we have quoted, he has found

$$a = 2{\cdot}37187,\ b = 3{\cdot}15312,$$

and the metre being unity in this measure, it gives for the preceding formula

[1] Recherches Physico-mathématiques sur la Théorie des Eaux Courantes, p. 73.

$$U = \frac{V\,(V + 2{\cdot}37187)}{V + 3{\cdot}15312}.$$

This formula has the advantage of representing with more fidelity, than that of Dubuat, the results of his own experiments, besides being in accordance with the phenomena in question, when considered in their proper bounds. M. de Prony remarks, that from $V = 0$, to $V = 3$ metres, the proportion $\frac{V + 2{\cdot}37187}{V + 3{\cdot}15312}$ is evidently equal to $0{\cdot}82$; and as practical cases are confined within these limits, the following formula will be found sufficiently exact:

$U = 0{\cdot}82.\,V$, or, which is nearly the same, $U = \frac{4}{5}\,V$. This is the total amount of all that theory and experiment have hitherto discovered as the means of ascertaining the amount of the mean velocity of a course of water by means of the velocity of the surface and the current; but something more is necessary to be observed respecting the manner in which the results we have explained may be applied to use.

The experiments of Dubuat have been made on artificial canals, the section of which was a rectangle or a trapezium, and in which the depth of the water varied from 54 millimetres to 27 centimetres. The results of these experiments must therefore be viewed with some reserve, and cannot be judiciously applied to the beds of rivers, as the nature of the motions of fluids is not sufficiently known to us as yet, so as to enable us to conclude on this point from small to great in a manner sufficiently accurate. Moreover, the experiments on

which the preceding formula is founded seem to indicate that the relations which exist between the three velocities, V, U, and W, are independent of the size and figure of the bed of the stream; and these relations seemed not to undergo any sensible change, where the breadth of the bed was six or seven times greater than its depth.[1]

It is therefore difficult, as M. de Prony observes, to persuade ourselves that these different elements have no influence on the relative values of V, U, and W; and experiments made on a larger scale, by throwing a new light on the subject, will compel us to have recourse to the conclusion already drawn from former observations.[2]

[1] Dubuat, Principes d'Hydraulique, tome i. p. 96.

[2] The author of these notes has published in the sixth volume of the Memoirs of the Academy of Sciences, the result of his inquiries on the motion of fluids, and has principally applied them to what is usually styled linear movement, that is to say, in the case where the fluid moves in a rectilinear bed, according to the lines drawn parallel to the axis of the bed. It follows from these inquiries, with regard to the question which we now investigate, that whatever may be the figure of the transverse section, the mean velocity, and the greatest degree of swiftness which takes place at the surface and the middle of the stream, tend to an equality between themselves, in proportion as the dimensions of the bed become greater or less. Besides, the proportion between these two degrees of velocity is independent of their absolute value. If, in a rectangular bed, the horizontal breadth is supposed to be extremely wide, and the vertical depth of the water extremely small, the mean velocity will be about 0·64 of the greater. If both the dimensions of the rectangular bed be extremely large, the proportion becomes nearly 0·41. The formulæ present the

At the period of great floods, which it is necessary to choose, in order to arrive at the knowledge of the elements of calculation for the quantity of water which is discharged by the river, the banks are overflowed, and the water spreads on both sides to a great extent, and flows very slowly; while at the same time it moves with great rapidity in the middle of the current. It is highly probable that great errors would arise in such cases as these, from the adoption of the mode of calculation already laid down.

We must therefore select, if possible, a spot where the waters of the river are enclosed, or where, during floods, the overflowing is not considerable. The section of the stream's velocity at the passage of a bridge might also be safely taken, should it happen that there might be a bridge in the vicinity of the locality selected for the intended erection.

It is obvious that the methods which we have hitherto employed to obtain directly the amount of the mean velocity of a river, are defective, and liable to lead to errors more or less considerable. As a certain analogy

means of calculating the values of this proportion for various beds, the transverse sections of which might happen to be semicircles or rectangles.

Besides, experience proves that the actual laws relative to the motion of water in the beds of rivers, or in tubes, differ from those of linear motion; and the only case in which Nature realizes the latter laws, is where fluids move in rectilinear tubes of a very small diameter. The results immediately deduced from observation, are, in the present case, alone deserving of attention.

must exist between the section, the slope, and the velocity of a current, and as it is always possible to measure the slope and the section, the value of the velocity might be naturally ascertained, if this analogy were sufficiently known. In the work which we have already quoted, M. de Prony has deduced it from a variety of experiments, and has arrived at the equation

$$U = -0{\cdot}0719 + \sqrt{0{\cdot}005163 + 3232{\cdot}96\,RI},$$

in which U being always the mean velocity of the current, R will represent the mean radius, that is, the area of the section divided by that part of the periphery of this section which belongs to the solid in which the fluid flows; and I represents the slope of declivity per metre.[1]

[1] Subsequently to the experiments of M. de Prony, M. Eytelwein has published, in the Memoirs of the Academy of Berlin, the result of some new experiments, by which he has laid down a formula like the preceding, the coefficients of which have values dissimilar in some respects. This formula, taking the metre for linear unity, is

$$U = -0{\cdot}03319 + \sqrt{0{\cdot}0011016 + 2735{\cdot}66\,RI}.$$

The experiments which served to determine the coefficients comprehended values from 2 to 3^m for the velocity, while those values in the experiments made by M. de Prony did not exceed $0^m{\cdot}88$. The formula of M. de Prony agrees at least as well as that of M. Eytelwein with the experiment for the small velocities, but it appears rather defective with respect to the greater ones.

M. de Prony had given, for the motion of water in tubes, the expression

$$U = -0{\cdot}02488 + \sqrt{0{\cdot}0006192 + 2871{\cdot}43\,RI}.$$

This agrees better than that which is quoted in the text with the experi

This equation will give the value of the mean velocity with sufficient exactness in every application, but it is necessary to remark that it proceeds essentially on the supposition that the extent of the section of the river, and the value of the declivity, are sensibly the same on a sufficient degree of length, so that the mean velocity therein may be regarded as uniform and regular; and it is requisite to pay attention to this consideration, whenever this formula is applied.

On the manner of fixing the outlet, or discharge, with reference to the mass of water in the river.

The dimensions of the beds of rivers are generally uniform and similar. But with the exception of particular cases, in which they flow through sandy soils that yield with so much facility to the action of the water, that the current may, at every increase, stray into different places, there is a certain equilibrium established in every point of the course of the stream, between the action employed by the waters on the stone work, and the tenacity of the materials of which the bed is composed; and in virtue of this equilibrium no very remarkable changes take place either in the extent or the form of

ments in what the value of the velocity is above one metre, yet it leads to results of rather an equivocal nature. See the work entitled *Recueil de cinq tables pour faciliter et abréger les calculs relatifs au mouvement des eaux*, &c., published by M. de Prony in September, 1825. The memoirs of M. Eytelwein have been translated, and printed in the *Journal des Mines*, tome xii. 1826.

the bed, which constantly preserves the same ordinary nature. But if, from the operation of any cause whatever, the force with which the water tends to wear the bottom and the banks of its bed should be augmented, the bed will be forced to enlarge itself till the equilibrium shall be established afresh, unless the sides are composed of matter that presents a more considerable resistance than the force with which they are assailed. If, on the contrary, the velocity of the river should have experienced a diminution, its bed would be raised up by successive deposits, till the extent of the section would become such as to resume, with the resistance of the bottom, the proportions marked by nature for the steadiness of the river's course.

It results from these principles, that if the breadth of a river were diminished by works erected on its bed, its velocity would be increased, and it would form a slack water and a declivity, or shoot; the water would react on the bottom, and the bed would deepen till the enlargement of the section, combined with the increase of the velocity, would be capable of making an equivalent for the diminution of the breadth of the river, and till this velocity should find itself in equilibrium with the resistance at the bottom. If, on the contrary, an attempt should be made to increase the breadth of the bed, the velocity would be checked, and the bed would have a tendency to be choked up.

From these considerations, it is a most essential point, in the construction of a bridge, to pay attention to the

velocity which the waters will assume under the arches; but this degree of velocity must not be augmented, to compel the current to react against the bottom of the river, and undermine the foundations of the piles and buttresses. Neither should it be sensibly diminished, because, in that case, the length of the bridge would be increased to no useful purpose, and this diminution would occasion deposits that might be dangerous in the end.

There are, nevertheless, some distinctions to be made with regard to the nature of the soil of which the bottom of the bed is formed. If it should be extremely compact and tenacious, and approach the nature of rock, it will not yield sensibly to the action of the water; and whatever may then be its velocity, no apprehensions need be entertained that the bridge will break down. The only thing to be observed here, is, that the waters cannot assume a great velocity without producing in the upper stream a slack water, more or less considerable, and which, in cases where the outlet would be extremely confined, might produce inundations in the upper part of the river, and render its navigation difficult and dangerous. If the bottom is composed of matter that the water can easily master, care must be taken not to permit the velocity to be sensibly increased; in short, if the soil yields to the action of the water with so much facility as to produce fears that during a flood the current would force its way under some of the arches, and penetrate the bottom, while it would deposit sand under

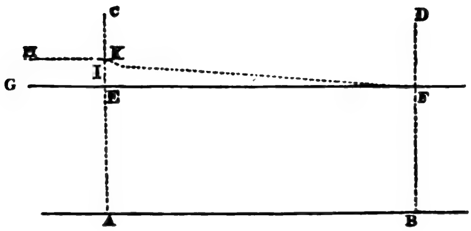
others, this would lead to the necessity of constructing a general frame of timber; then the bottom, offering in every direction the same resistance to the action of the current, would oblige it to assume a regular course, and distribute itself equally over the whole breadth of the river. As long as this frame should exist, no apprehensions need be entertained that any of the piles will be undermined.

It is evident, from the preceding remarks, that the velocity which the water may take under the bridge ought, in all cases, to be determined beforehand, either from the nature of the soil which composes the bottom of the river, or from the elevation of the slack water or eddies formed in the upper stream, on which the value of this velocity depends; and as the quantity of water impelled by the river is likewise known, the superficial extent of the outlet which the bridge shall have may be immediately established. The question is, therefore, reduced to the following problem: the section of the bed of a river, and the velocity of the stream, being given, to determine the mean velocity which the current shall acquire, and the elevation of the slack water or eddies, on the supposition that the bed is narrowed by the construction of the piers.

This problem is not susceptible of a rigorous solution, but we shall show, by neglecting some circumstances, the effects of which are not very sensible, and which compensate one another in a great measure, and by following the example of Dubuat, we shall obtain an

approximate solution which may be employed to great advantage in cases of this kind.

Suppose ACDB to represent the lateral face of a



pile, and GEF the natural inclination of the river before the construction of the bridge. As the current is confined within the interval EF, the velocity, and consequently the inclination of the river, will be in that part more considerable, and the surface of the water, omitting the particular resistances produced by the starlings, will take a direction that may be represented by the line IF. This surface, in the upper stream, will necessarily rise above the point I, and we designate it by the line HK, which is sensibly horizontal to a length of no great extent.

Let us call

Ω, the natural section of the river;

ω, the area of the section after the construction of the bridge, or the extent of the outlet;

V, the mean velocity of the water;

v, the mean velocity of the water under the arches, after the construction of the bridge;

I, the inclination of the river in metres;

s, the length of the piles and buttresses = AB or EF;

H, the elevation of the slack water = EK;

g, the accelerating force of gravity $= 9^{m} \cdot 809$.

We shall then have $v = \frac{\Omega}{\omega} V$, since, in any river, the velocities are in the inverse ratio of the area of the corresponding sections; and the heights, due to the velocities V and v, will be represented by

$$\frac{V^2}{2g} \text{ and } \frac{\Omega^2}{\omega^2} \frac{V^2}{2g}.$$

The part IK of the elevation of the slack water which corresponds with the augmentation of the velocity, will be

$$IK = \frac{V^2}{2g}\left(\frac{\Omega^2}{\omega^2} - 1\right);$$

and as, by reason of the contraction generally met with by the current at the passage of a bridge, the section ω is diminished, we replace in this expression the area ω by $m\omega$, designating by m a coefficient, the value of which depends principally on the size of the piles, as well as on the form of the starlings and the formation of the arches. We shall then have

$$IK = \frac{V^2}{2g}\left(\frac{\Omega^2}{m^2\omega^2} - 1\right).$$

It will now be necessary to find the value of the inclination, which will be found by the length of the piles. Before the construction of the bridge the inclination was equal to sI, and as these inclinations increase nearly in

proportion to the corresponding velocities, we shall have for the value of the new inclination

$$sI\,\frac{\Omega^2}{m^2\omega^2}.$$

Thus the part of the elevation of the slack water which arises from the increase of the inclination under the arches of the bridge will be represented by

$$EI = sI\left(\frac{\Omega^2}{m^2\omega^2} - 1\right);$$

and we shall then have

$$EI + IK = H = \left(\frac{V^2}{2g} + sI\right)\left(\frac{\Omega^2}{m^2\omega^2} - 1\right)$$

for the elevation which the waters of the river assume in the upper stream, since their level must remain the same in the lower.

In order to apply the result which we have obtained, it will be sufficient to put in the place of V the expression $v\,\frac{m\omega}{\Omega}$, and to give afterwards to v the value that is determined beforehand, after the principles laid down above for the velocity which the waters ought to take under the bridge.

With respect to the value of the coefficient m, we are far from being able to determine it with a desirable degree of exactness. The experiments made relative to this matter have been performed on orifices excavated in walls of different thickness, to which tubes were sometimes annexed, through which the water flowed away under a pressure of greater or less force. Besides, the

dimensions of these orifices, even according to experiments made on a large scale, are not to be compared with those of the arches of a bridge, and the circumstances of the vent or discharge are different from those that take place in the case that occupies our attention.

It will be afterwards seen, when we investigate the most advantageous form to be given to the starlings of the piles of bridges, that this form has a very sensible influence, as well as the thickness of the pile, on the contraction that takes place, and by which the natural flow of the river is modified when it comes in contact with the bridge. The value of the coefficient m, as we have observed before, depends principally on the form of the starlings, and the projections of the arches, when they are sunk under the water. We cannot assign with sufficient exactness that which it ought to assume in different cases, but we shall not be far from the truth in supposing $m = 0{\cdot}95$ when the piles terminate in a semicircle, or in acute angles; $m = 0{\cdot}90$ when they terminate in obtuse angles; $m = 0{\cdot}85$ when they terminate in right angles, supposing the arches to be wide. In the most disadvantageous cases, that is, in the case of small arches, and where the projections of the arches are sunk in the water, the value of the coefficient m will perhaps be about $0{\cdot}7$. By putting in the preceding equation for H and v their values found from the nature of the soil and other local circumstances, it will be easy to deduce that of the proportion of the two sections ω and Ω.

In order to determine beforehand the elevation of the

slack water that may be formed in the stream above the bridge, it will be requisite to anticipate the changes that may arise in the elevation of the waters of the river, and the inundations resulting from it that may overflow the banks. According to the principles laid down by Dubuat, it is generally admitted that the surface of the water, which, previous to the construction of the bridge, and on the hypothesis of a uniform inclination, has a manifest tendency towards an inclined plane, becomes, after its construction, a concave surface, which comes into contact with the primitive surface where the rising of the water ceases. This author has even given rules for the calculation of the slack water, on the supposition that the section of this surface would form an arc of a circle of great magnitude. But on these investigations we cannot place implicit reliance. When the figure of the bed of a river is irregular, and presents considerable variations in the declivity of the bottom and its breadth, where, in the event of a flood, a part of the waters flow slowly, and on inundated banks, the exact determination of the figure assumed by the surface of the fluid cannot be obtained. But we can arrive at it with a sufficient approximation when the bed of the river is regular, and the motion of the water proceeds in a uniform manner. M. Bélanger, engineer of roads and bridges, has furnished a calculation in the following manner for this particular object. Let us call s a length computed on the bottom of the bed, in the direction of the stream; h the vertical height of the transverse section of the current; ω the area of this section, corresponding to the height h; x the extent of the

section ω, taken at the surface of the water ; R the mean radius ; i the inclination of the bottom of the bed, which is supposed to be uniform ; Q the volume of water discharged by the river in a second ; and we have the equation

$$s = \int dh \, \frac{\dfrac{Q^2 x}{g\omega^3} - 1}{\dfrac{1}{R}\left(0{\cdot}00002427 \, \dfrac{Q}{\omega} + 0{\cdot}0003655 \, \dfrac{Q^2}{\omega^2}\right) - i},$$

in which the metre and the sexagesimal second are taken for unities of length and time, and in which g represents the velocity $9^{m}{\cdot}809$, generated by gravity in one second. All the quantities comprised under the sign $\int$ may be estimated as a function of the elevation of h in the section. In calculating the value of the integral indicated by this sign, setting out from the highest value of h, which takes place at the point of the rising of the stream, that is, immediately in the wake of the bridge, till we arrive at any fixed value whatsoever, which may be intermediate between the foregoing and the natural value of the section, the result will give the length comprised between the part of the stream above the bridge, and the section to which this second value of h belongs. We may thus arrive at the knowledge of the relations between the given elevations of the sections, and the distances from the bridge where these elevations take place, and consequently, of the figure of the fluid's surface throughout the whole extent of the slack water. This extent is usually prolonged, to speak rigorously, to an infinite distance, but the rising of the stream ceases

to be evident at a limited distance.[1] Besides, the formation of slack water at the upper part of a bridge must occasion difficulties either with respect to the solidity of the structure, or the obstacles opposed to the navigation of the river ; it ought therefore to be diminished by every possible means.

It has been observed above, that it is dangerous to give a river too wide an outlet, because in this case it might give rise to masses of alluvion, which, acquiring in time consistency enough to resist the action of the current, would, during floods, force the waters to pass under the arches that are less exposed to this obstacle, and occasion the piles to be undermined. For a similar reason, a bridge ought not to be composed of two parts separated by an island, as it is probable that one of the parts having been choked up, the whole current would be forced to flow to the other, which would lead to the destruction of the bridge. It was by an accident of this kind that the bridges of Chazey and Roanne were swept away. It may be additionally remarked, that bridges, in general, are never destroyed unless by some error in the outlet, and that ultimately too great a diminution of the section is always the cause of their ruin, either when a sufficient length has not been allowed to the bridge in the first instance, or, on the contrary, where a too considerable one has been given.

All the elements for calculating the area of the outlet

[1] It will be necessary, on this subject, to consult a publication by M. Bélanger in 1818, entitled "*An Essay on the numerical solution of certain problems relating to the permanent motion of running waters.*"

ought to be assumed at the period of great floods, and it is according to the quantity of water discharged at that time that this area should be determined. It must be observed, however, that it is expedient to arrange the arches (in rivers susceptible of this method) in such a manner that, during the lowest state of the current, there should be the depth of one metre under some of them, so that the navigation of the river shall not be impeded. It will be always possible to combine the different conditions of which the size and form of the arches may admit, and to arrive, in each particular case, at the best solution of the problem.

The preceding notions are easily applicable to the bridge now building over the Durance, at Bonpas. This bridge, which is of wood, is built between two mounds at a distance of 534 metres from each other. The breadth of the river, at low water, is only 110 metres, and its mean depth $1^{m}\cdot30$. But, during floods, the stream rises to 3 metres, and then the surface of the outlet is 1530 square metres.

As the bed of the river is not confined, and its breadth is considerable relatively to its depth, it was difficult to ascertain its mean velocity with any degree of exactness. But at the distance of about 4 myriametres above Bonpas, the Durance passes between two rocks, in shallow water, that are distant 180^{m} from each other. The elevation of the stream, at this spot, has been observed during considerable floods, and has been found to be :

1st, That the river having $2^{m}\cdot44$ of reduced depth, its mean velocity was $1^{m}\cdot95$ a second.

2nd, That when the river had $2^{m}\cdot92$ of reduced depth, the mean velocity was $2^{m}\cdot44$ a second.

3rd, That at the period of great floods, the depth of the water was $4^{m}\cdot87$, and its mean velocity $4^{m}\cdot12$. Thus, in the latter case, which is most worthy of observation, the discharge of the river is from $180\times4^{m}\cdot87\times4^{m}\cdot12=$ 3612 cubic metres a second.

As the distance between Mirabeau and Bonpas is not considerable, and the river receives, between these two points, only some streamlets or torrents of no magnitude, there cannot be any great difference between the quantity of water that flows to both. But we may determine this difference by approximation, by comparing the surfaces of the basins that supply water to Mirabeau and Bonpas, where we shall find that the latter surpasses the other by $\frac{1}{16}$. Thus, at Bonpas, a volume of water passes in a second equal to 3838 cubic metres, which, in a section of 1530 square metres, gives a mean velocity of $2^{m}\cdot51$. Though the bridge is built of wood, it does not diminish, but in a slight degree, the superficies of the outlet; and it is observable that the velocity of $2^{m}\cdot51$ a second, which corresponds with an elevation of the water of more than 3^{m} in the bed of the river, scarcely surpasses $2^{m}\cdot44$, the velocity which the river naturally takes at Mirabeau for an elevation of $2^{m}\cdot92$. Thus the mean velocity at Bonpas is less considerable than in other parts of the river's course, and its waters move with great facility.

The outlet of the Rhône at the bridge of St. Esprit is nearly 3580 square metres, that is to say, a little more than double that of the Durance at Bonpas, while the super-

ficies of the basins that supply the Rhône at St. Esprit is five times greater than that of the basins that supply the Durance at Bonpas. Thus, though the velocity of the Rhône is very considerable under the arches of the bridge of St. Esprit, yet it appears that the velocity ascribed to the Durance at Bonpas is too great. But it will be observed that the latter river, at Bonpas, at no more than 25 myriametres from its source, is still a torrent, while the Rhône, at St. Esprit, is no longer so.

It is therefore a very essential point, in determining the vent or outlet of bridges, to distinguish rivers from torrents, and to bestow strict attention on the superficies of the basins, the nature of the soil, and the time which the water takes to pass. It may happen, as we have observed before, that it is necessary to construct a bridge of greater extent at a short distance from the river's source than at a place more remote from it, though it may have received many accessory streams in the interval.

III. ON THE FORM OF THE ARCHES.

The arches of bridges are divided, with respect to their form, into three principal kinds, viz.: arches of a full semicircle, described by half of the circumference; arches of a flat vault, usually described by several arcs of a circle of different radii, the form of which is nearly similar to a semi-ellipse; and arches of a circular kind, which are formed from arches of greater or less magnitude.

Semicircular arches were anciently most in use. There are but few ancient bridges in which this form is not used, which for a long time has prevailed in Europe, and which has the advantage of presenting most solidity and facility in the construction. However, arches of this form are subject to the inconvenience of considerably obstructing the passage of the water.

The use of flat-vaulted arches was not introduced into France till the close of the seventeenth century. What led to the adoption of this form was the necessity of affording a wide discharge without considerably augmenting the height of the arches. It effectually answered this object, and when the two diameters are not very unequal, it presents as much solidity and facility in the construction as the full semicircular arch.

With respect to arches formed of circular arcs, it is necessary to make a distinction in the different cases of this form. The first is that in which the springings are under water, as is the case with most of the first great bridges built in France, such as the bridge of St. Esprit, and the ancient bridge of Avignon. In this case, the flat-vaulted arches have the disadvantage of giving a less considerable discharge, and admitting of very massy tympana. This latter defect seems to have been recognised by the first builders, for the backs of the arches are almost always filled simply with earth, or relieved by means of small arcades.

In the second case, the springings of the arc are raised perpendicularly nearly on a level with the highest water of the river, as may be seen in the bridge of Louis XVI.

at Paris, a circumstance which compels the architect to make the arc very low. Hence it happens that the lateral pressure of the voussoirs is very considerable, and it is necessary in such an instance to employ great care in the construction, so that the crown may not be subject to sink down after the arching, as sometimes happens. The manner in which this arch juts out is different from that which takes place in others, but such irregularities do not tend to overturn the piers, only giving them a horizontal declination. We shall hereafter point out the means which are most suitable for resisting this pressure without any extraordinary expense.

The resistance opposed by the piers to the current, when they are sunk, is one of the principal causes of the alluvions that are formed under the piles. Bridges built on the principle of the arc of a circle have, in this respect, a great advantage over the others, when the springings of the arches are not reached by the waters of the river.

It is hardly possible to lay down any general rules for the choice that is to be made of the different kinds of arches, as the decision, in each particular case, must depend on the various localities that may occur. The extent of the outlet which should be given to the river, the relative heights of the water at the greatest and lowest elevations, the height at which the surface of the pavement of the bridge can be placed, the obligation which is sometimes imposed, of allowing for the destruction of an arch, and consequently of making the

piles perform the office of abutments, are the principal circumstances that lead to a determination on this subject.

To the three kinds of arches of which we have spoken, and which are the only ones now in use in France, we must add the forms employed by the Arabians, and especially the Gothic form, composed of two arcs of a circle, and known by the name of ogee. The latter would have the inconvenience of diminishing considerably the outlet, a fault which is easily corrected by working apertures in the tympana, as has been done in one of the bridges of Pavia. There may be cases in which this form is to be preferred, but taste ought to proscribe none, as all of them possess merit when suitably applied.

IV. OF THE SIZE OF THE ARCHES.

Although the size of the arches usually depends on the particular circumstances of the locality of the intended bridge, yet we shall venture to sketch out a few ideas that may elucidate this point.

Small arches are best suited to quiet rivers, the waters of which do not rise to any considerable height. It is, in that case, easy to fix the foundations, and it furnishes an additional reason for not fearing to multiply the points of support. Large arches, however, are most suitable to torrents, in which it is generally difficult to lay the foundations, and in which the waters frequently draw down rocks and trees, which may damage the piles and the

abutments of the arches, which, in that case, ought to be placed above the level of the water.

In large rivers, wide arches ought in general to be adopted in preference, especially when they are subject to inundations, or sudden risings of the stream; but the method, whether more or less expensive, used in forming or establishing the foundations, will influence this determination. Regard must be also paid to the nature of the materials to be employed in the construction of the bridge, which ought to afford more solidity for large arches than small ones, and also to the nature and size of the vessels that navigate the river, which will require a free and convenient passage.

With respect to the width to be allowed to arches, there are two plans to be pursued: the apertures may either be all equal, or they may progressively diminish from the middle arches to those that join the abutments.

If all the arches are equal, there is the advantage of giving to the tops of the vaults the same elevation above the water, and of using the same centering which served for the two first. But then it is necessary to raise the approaches to the bridge, and it may consequently become requisite to make considerable embankments, and possibly to endanger the houses that may be in the vicinity. The rain water that is deposited for a length of time on the bridge is not easily got rid of, as it penetrates down to the coat, and undermines it.

When the diameters of the arches are unequal, these latter inconveniences disappear, and we are then at liberty to give to each side of the pavement an inclination

which ought not to exceed three centimetres per metre.[1] We thus diminish the obstacles to the approaches, and the height of the embankments. It is, moreover, possible to unite the advantages of both these methods, by giving the same width to all the arches, and placing the springings at heights decreasing from the middle to the extremities of the bridge.

It is necessary to give a sufficient height to the arches, so that in floods, extraneous matter, such as trees, carried down by the stream, may pass freely under them. The minimum of this height, when the arches are equal, is nearly one metre; when they are unequal, the height lies between 70 centimetres and $1^{m}\cdot 4$.

V. OF THE BREADTH OF BRIDGES.

The breadth that ought to be allowed to bridges depends wholly on the locality in which they are situated. It ought to be regulated by the degree of importance attached to the road on which they are built, or the population of the town which they benefit, and it is an essential point not to make this breadth too considerable, as it leads to a superfluous expense.

If the bridge is built in the country, and for a neighbouring road, it will be sufficient to allow it from 4 to 5^{m}, especially if it is not very long. For a road of the second class, the breadth ought to be from 6 to 7^{m},

[1] This is 3 in 100. See Table of English and French Measures, p. 39.

which will allow room at once for two carriages, and for foot passengers, to pass. An extent of from 9 to 10 metres ought to be allowed to a bridge on a road of the first class.

In the interior of cities, the breadth of the bridge may vary from 10 to 20 metres, with respect to the population, and the activity of commercial intercourse, but it ought not to exceed the latter limit. The Pont-Neuf, at Paris, is undoubtedly one of the greatest thoroughfares in the world, and its road-way, which has a breadth of 22 metres between the parapets, is by no means too confined.

NOTE

On the degree of velocity necessary to currents to support and convey different matters.

We shall quote beforehand some estimates of the velocity of currents, observed under different circumstances, which may furnish some terms of comparison.

The ordinary velocity, per second, of the small rivers in the vicinity of Paris, the declivity being 0·00018	0·28 per metre.
The velocity of the Seine between Surène and Neuilly, observed by M. de Chézy, the elevation over the low water being $1^m \cdot 26$, and the declivity 0·000125	0·78
The velocity of the Seine, in the interior of Paris, the water being at $0^m \cdot 6$ at the bank, and the declivity 0·00055	1·00
The same, the water being 6^m at the bank, and the declivity 0·0006	1·90
The greatest velocity of the Thames, at London, during high water	0·90
At low water	0·76
The velocity of the Tiber, at Rome, at low water . .	1·00
The velocity of the Danube, at Ebersdoff, at low water	1·05
At high water, this velocity varies from $2^m \cdot 21$ to $3^m \cdot 79$.	
The velocity of the Loire, the declivity being 0·000382	1·30
The velocity of the Rhône, at Arles, at low water .	1·46
The velocity of the Rhône, at Beaucaire, the same time	2·60
The ordinary velocity of the Durance, from Sisteron to its mouth, the height of the water at the bank not exceeding 3^m	2·60

The velocity of the Maragnon at the pass of the Pongo, observed by M. de la Condamine . . . 3·90 per metre.
The velocity of a torrent arising from the melting of snow caused by the eruption of a volcano, observed in America by Bouguer . . . . 7·80

We now insert a table, furnished under the article "Bridge," in the Edinburgh Encyclopedia, by Messrs. Telford and Nimmo, indicating the velocity of different currents, and the nature of the materials likely to yield to their action.

The ordinary nature of currents.	Velocity per second.	Materials that resist these velocities, and yield to more powerful ones.
	Metres.	
Very slow	0·076	Wet ground, mud.
Gliding	0·152	Soft clay.
Gentle	0·305	Sand.
Regular	0·609	Gravel.
Ordinary velocity	0·914	Stony.
Extraordinary and rapid floods	1·022	Broken stones, flints.
Extraordinary floods, and rapids	1·052	Collected pebbles, soft schistes.
	1·083	Beds of rocks.
Torrents and cataracts . . .	3·005	Hardened rocks.

We shall next insert the results of the experiments made by Dubuat, in order to discover the regular velocity that best suits soils of different natures, the materials of which are placed at the bottom of an artificial canal formed of thick planks.

Velocity per second.	Materials that resist these velocities, and yield to more powerful ones.	Specific gravity.
Metres.		
0·081	Brown clay, fit for pottery	2·64
0·108	Large gravel, like a grain of aniseed	2·54
0·162	(The preceding materials are carried away.)	
0·189	Gravel as large as a pea, at most	the same.
0·217	Large yellow sand	2·36
0·325	Gravel as large as a Marsh-bean	2·54
0·474	(The preceding materials are carried away.)	
0·650	Round pebbles of the sea-shore, of 0^{m}·027 diameter, at most	2·61
0·975	Angular flints, as large as a pullet's egg	2·25
1·220	(The preceding materials are carried away.)	

From these experiments it may be inferred, that the materials most commonly found in the beds of rivers, such as sand and gravel, are carried away by very low velocities. It follows that these materials are, in general, in a constant state of motion. They flow away, as well as the water, but more slowly, and by a species of particular change of place, which has been well observed and described by Dubuat. But it is not necessary, in order that a bridge may not be undermined, that the velocity of the water under the vaults shall not surpass the velocity of the current that may carry away the materials which form the bottom of its bed; but it is necessary that this velocity shall not sensibly surpass that which takes place above and below the bridge.

ENGLISH AND FRENCH MEASURES.

ENGLISH.	FRENCH.
Inch (36th of a yard)	2·539954 centimètres.
Foot (⅓ of a yard)	3·0479449 décimètres.
Imperial yard	0·91438348 mètres.
Fathom (2 yards)	1·82876696 mètres.
Pole or perch (5½ yards)	5·02911 mètres.
Furlong (220 yards)	201·16437 mètres.
Mile (1760 yards)	1609·3149 mètres.

FRENCH.	ENGLISH.
Millimètre	0·03937 inches.
Centimètre	0·393708 inches.
Décimètre	3·937079 inches.
Mètre	39·37079 inches. 3·2808992 feet. 1·093633 yards.
Kilomètre	1093·63 yards.
Myriamètre	6·2138 miles.

SYSTEMATIC NAMES.	VALUE.
ITINERARY MEASURES.	
Myriamètre	10,000 mètres.
Kilomètre	1000 mètres.
Décimètre	10 mètres.
Mètre	The basis of the weights and measures. Ten millionth of the quadrant of the terrestrial meridian.
LONG MEASURE.	
Décimètre	10th of a mètre.
Centimètre	100th of a mètre.
Millimètre	1000th of a mètre.

ON THE DESCRIPTION OF THE ARCHES OF BRIDGES.

VI. ARCHES OF A FULL SEMICIRCLE.

We have already explained the principal advantages and inconveniences of arches of this kind. No further remark is necessary with respect to their description, as their form is entirely determined when the aperture is settled, since this form is a semicircle of which the aperture is the diameter. It is sufficient to observe that the centre of the semicircle is usually situated at the elevation of the foundations, or that of the low water. When circumstances admit of it, it may be placed below this elevation, and then the arch is perpendicularly erected.

VII. ARCHES WITH FLAT VAULTS.

This form of an arch is not entirely determined when the span, and even the elevation, have been already fixed. It is, in fact, possible to describe on two given diameters an infinite number of different curves.

The only conditions to which the curve of flat-vaulted arches is subjected, are, that the tangent at the vertex should be horizontal, and that the tangents at the springings should be vertical. As the semi-ellipse satisfies both these conditions, it seems natural to choose this curve, especially as the curvature decreases regularly from the springings to the vertex, and forms an agreeable object to the eye. But it has the inconvenience in the construction of obliging the architect to change the form of the voussoirs that compose the arch, a circumstance which is very unpleasant, and has, besides, the defect of not allowing so wide an outlet as the curves of which we are about to treat, unless the difference between both the diameters of the arch be not very considerable.

Instead of the semi-ellipse, curves composed of a certain number of arcs of circles are usually employed, because the builder is then at liberty to determine conveniently the length and the radius of these arcs, and to give to the flat-vaulted arches the form which may be found most suitable and convenient.

In this case, both of the following conditions are necessary to be observed, viz.; 1st, that the line of the first arc, setting out from the abutments, shall include that of the ellipsis on the two diameters of the flat-vaulted arch, so as to afford to the arch a wider scope of outlet than it would have without this ellipsis; 2nd, that the radius of the vertical arc shall not exceed a certain limit. But the value of this limit cannot generally be determined, though it should scarcely

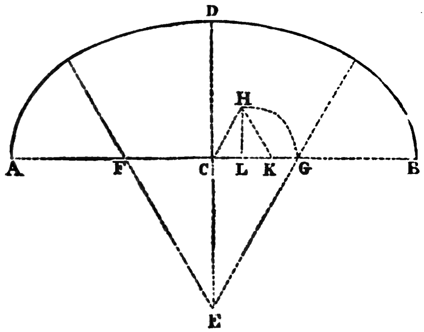
exceed the span of the arch; and if it be found necessary, from particular circumstances, to employ a greater radius, care must be taken not to direct the joints of the arch-stones to the centre of the arc, but towards a nearer point.

After settling these two first conditions, the next step will be to fix the number of arcs that are to enter into the composition of the flat-vaulted arch; they must not be less than three, and it has never happened that more than eleven have been used.

We shall now explain the mode of description in all these different cases.

Flat-vaulted arches described with three arcs of a circle.

The length of the radii of the three arcs that are to compose the flat-vaulted arch not having been fixed when we determined the two diameters of the curve, another condition must be employed. But it will be first necessary to suppose that the three arcs make each sixty degrees, that is to say, each of them equal to the sixth part of the circumference.



Let the half of the aperture of the arch $AC=a$, and its elevation $CD=b$. The centres of the two arcs described from the springing of the arch-stones will be then situ-

ated in the points F, G, in the line AB, and the centre of the third arc in the point E of the line DC produced. One of the centres being found, the two others will be easily determined. Let us call x the distance of the point C, the middle of AB from the centre G, then as the triangle FGE is equilateral, EG=FG, and consequently,

$$CE+CD=FG+GB,$$

in which

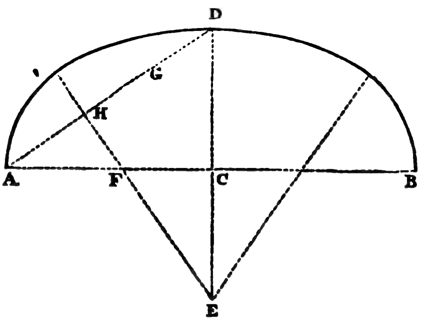
$$\sqrt{3}.\,x+b=x+a,$$

and resolving this equation,

$$x=\frac{a-b}{\sqrt{3}-1}=\frac{\sqrt{3}+1}{2}(a-b).$$

To construct this equation take $CK=a-b$, and upon it describe the equilateral triangle CHK, let fall the perpendicular HL, and from L apply LG=LH the altitude of the triangle, and G is the required centre.

Let us now suppose a condition to be laid down that the radii of the great and the small arc are different in the least possible degree from each other. Let $AC=a$, $CD=b$, the radius of the arc at the abutment y, and x that of the vertical arc, the centre of which is in E, and we shall have, by means of the right-angled triangle CFE,



$$(x-y)^2=(x-b)^2+(a-y)^2.$$

By solving this equation for x, we find

$$x = \frac{\frac{1}{2}(b^2+a^2)-ay}{b-y};$$

we shall then have for the expression of the proportion of the two radii,

$$\frac{x}{y} = \frac{\frac{1}{2}(b^2+a^2)-ay}{by-y^2};$$

by making its differential equal to zero, and finding the value of y, we have, after putting, for the sake of conciseness, $b^2+a^2=c^2$,

$$y = \frac{bc}{c+(a-b)};$$

this substituted in the above, gives

$$x = \frac{ac}{c-(a-b)}.$$

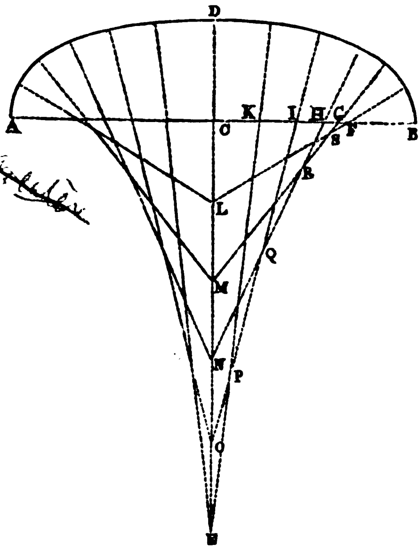
These two expressions are constructed by taking from the line AD the distance DG equal to $a-b$, and raising on the middle of the remainder AG, the perpendicular HE; then the points E, F, at the intersection of the two diameters will be the centres sought. This method gives a greater difference between the extent of the two arcs than the foregoing one, which appears preferable.

When the proportion between the two diameters is not less than one third, the difference of the radii of the arcs of which the flat-vaulted arch is formed is not great enough to make the transition from the one to the other too visible, and to produce a disagreeable effect; in that case it will be sufficient to describe it with three arcs of a circle. But when the arch is lower, then it will be necessary to employ a greater number.

Of flat-vaulted arches described with more than three arcs of a circle.

Different methods have been adopted in order to determine the positions of the centre, and the length of the radii of the arcs of which these curves are to be composed. We shall explain the mode which was preferred for the plan of the arches of the bridge of Neuilly.

After having determined the radius FB of the first arc, setting out from the abutments, a distance was taken for the prolongation of the less diameter; call this distance CE, which is arbitrarily made equal to three times CF, and which may have a different proportion to this line. Afterwards dividing CE into five equal parts, CF into five parts in proportion with the numbers 1, 2, 3, 4, and 5, and joining the points of division by the lines LF, MG, NH, OI, EK, the points E, P, Q, R, S, F, which are found at the respective intersections of these lines, were assumed for the centres of the different arcs that compose the flat-vaulted arch.



It is evident that in the curve described in this man-

ner the proportion of the elevation CD at the aperture A B, depends on the foregoing data, assumed from the length of the line CF, and its proportion with respect to CE. But, when a flat-vaulted arch is to be described, the two diameters are usually determined beforehand, so that, after describing a curve by the preceding method, it will be requisite to modify it in such a manner that the elevation CD shall become precisely equal to the one that is assumed.

Let us call half of the aperture in question a, and its elevation b, x the extent which CF is to have, and y that which is to be assigned to CE. Let us suppose, besides, that the assumed and arbitrary values given to CF and CE be represented by n and m, and that these results form, for the developement of the portion of the polygon EPQRSF, a length equal to s, while the length really to be described on the lines x and y will be equal to z.

We then have the relation

$$z+a-x=y+b\,;$$

and if we suppose the figure constructed on the lines x and y, to be similar to the figure ECF constructed on the lines CF$=n$ and CE$=m$, we shall have

$$y=\frac{mx}{n},\ z=\frac{sx}{n}\,;$$

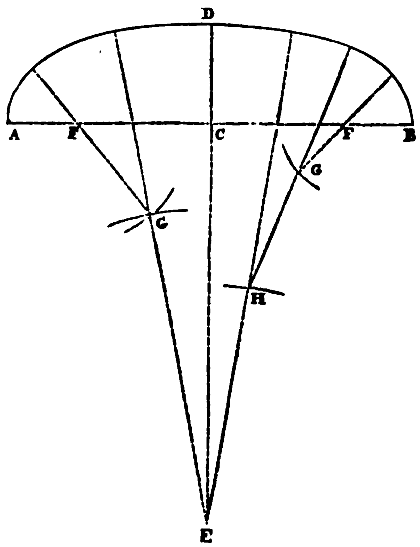
substituting these values in the foregoing equation, and deducing that of x, we find

$$x=\frac{n\,(a-b)}{m+n-s}\,:$$

this is the value that must be given to CF, in order that the aperture and the elevation of the flat-vaulted arch shall be precisely equal to the lines represented by *a* and *b*.

It is plain that this method may be applied to flat-vaulted arches, composed of any number of arcs whatever. By means of the same centres parallel curves may be described, in which the proportion of the diameters will be variable, and if the proportion between the distances CF and CE be suitably settled, then curves will be found which, described for the same arcs, will furnish forms of great variety, and allow a more or less free passage to the water.

The foregoing method has no inconvenience but its length, and leaves little to be desired; but it is imagined that in the case in which the arch should be lowered one-fourth it is fruitless to compose the flat-vaulted arch with a great number of arcs of a circle, and that in general it will be sufficient to confine their number to five; in this case the description is simplified in a great degree.



Let us suppose that the lengths AF and DE, the radii *r* and R of the arc at the buttresses, and the vertical,

are previously determined. Let us call the radius of the intermediate arc ρ; we can determine it by the condition to be a mean proportional between r and R; we shall then have $\rho = \sqrt{R\,r}$. Then describing from the point F as a centre, and with a radius equal to $\rho - r$, an arc, and from the point E, and with a radius equal to $R - \rho$, a second arc intersecting the former in G, the point G shall be the centre of the arc which will reunite that at the buttresses with the vertical one.

If it should be necessary to describe the flat-vaulted arch with seven centres, it will be proper to find two mean proportionals ρ and ρ' between the radii of the extreme arcs r and R, which will give

$$\rho = \sqrt[3]{Rr^2} \text{ and } \rho' = \sqrt[3]{R^2r}.$$

Then describe from the point F as a centre, and with a radius equal to $\rho - r$, an arc which will contain the centre of the arc of which ρ is the radius; and from the point E, with the radius $EH = R - \rho'$, a second arc which will contain the centre of the arc of which ρ is the radius. In order to fix afterwards on each of these arcs the respective positions of the two centres, it will be necessary to draw between both these arcs a line HG, the length of which shall be equal to $\rho' - \rho$; but, as the position of this line is not determined by this condition only, it must be fixed by means of a tact on the part of the architect that may be guided by the condition, that the extent of the arcs of which the flat-vaulted arch is to be composed shall decrease gradually and uniformly from the vertex to the springing.

This method may be extended to flat-vaulted arches composed of a greater number of arcs; but it is always nearly useless to employ more than five.

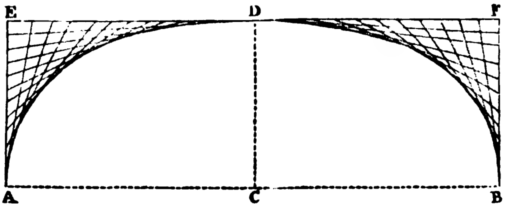
When an extensive combination of arcs is to be formed, it is not possible, unless in the parts near the springings or abutments, to employ the beam-compasses to strike the necessary arcs. The mode is, to begin by fixing the extremities of each of these arcs, the co-ordinates of which have been calculated beforehand, in such a manner as to form an angle, the supplement of which shall be equal to the half of the arc. This angle must be moved in such a way that its sides shall always pass through the extreme points of the arc, the vertex of which gives the intermediate points.[1]

Of flat-vaulted arches which are not described with arcs of a circle.

The difficulty of tracing out on a large scale, and in a manner perfectly exact, the curve which is projected, when it is composed of several arcs of a circle, has given rise to different methods of describing the flat-vaulted arch, in which this difficulty almost totally vanishes.

Carpenters usually employ, in order to adapt a curve

[1] For a full description of this as well as different other methods of describing arcs of a circle, see Dr. Gregory's Mathematics for Practical Men, pp. 127, 128.



to two sides of an angle AED, a method which consists in dividing the two sides of the angle into a number of equal parts, and to join the points of division by lines which they consider as tangents to the curve, and which, being supposed to approximate infinitely, determine each of their points by their successive intersections. By performing this same operation on the angle BFD, a portion of a curve will be found equal to the first, and which will complete the description of the arch ADB.

It has been observed for a length of time that the curve described in the foregoing manner was a portion of a parabola, the vertex of which is situated between the points A and D; and it allows of a wider outlet than is afforded by a flat-vaulted arch composed of three arcs of a circle, or by a full semicircle, which should be described on the same axes. It thus presents an advantage in this respect as well as in the facility of description. It retains however an inconvenience resulting from the more or less agreeable aspect which curves of this kind present; in fact, it is evident that in the parabola the value of the radius of curvature is a *minimum* at the vertex of the curve; and by reason of the plane where the vertex is fixed in this instance it follows that the curve does not diminish progressively from the springings to the most elevated point of the

arch. It has besides been proposed to form the flat-vaulted arch from two arcs of a circle described from the line of the springings, and connected at the vertex by a portion of the catenary curve. Curves composed in this manner present, particularly in the lower part, a much wider outlet than the ordinary flat-vaulted arches, and in this respect they seem to be deserving of a preference.

VIII.—ARCHES FORMED FROM AN ARC OF A CIRCLE.

Latterly a very considerable number of bridges has been constructed in which the sweep is described by an arc of a circle, but care has been always taken to place the springings nearly on a level with high water. Their positions being fixed by this condition, if the height to which it is possible to raise the vertex of the arch is equally afforded by the localities, the arc is then completely determined. But very possibly, in this case, the opening would be too low, or the radius of the arc may have too great a length, so that the work would not present that solidity which is requisite to be obtained in such a construction as a bridge. In that case, it will be best to abstain from the use of the arc of a circle, and from the advantages which, in other points of view, it might afford.

IX. — ON THE SUBSTANCE WHICH SHOULD BE GIVEN TO THE ARCH-STONES OF BRIDGES.

After having determined the curvature of the arches, the next question which presents itself is that of the depth of the stones of the arches at their crown; because it is upon this that the extent and the direction of the thrust depend, and consequently the resistance of which the piers and abutments should be capable. In regard to this point the works of the ancients, and even those of the moderns, exhibit very great differences. Several engineers have, indeed, endeavoured to establish precise rules for this purpose, yet as such rules are not founded upon true and obvious principles, they have not been strictly adhered to. We shall now proceed to state some particulars briefly, on those which are most generally known.

The principal Italian architects, such as Alberti, Palladio, and Serlio, have vaguely indicated a few of them: one having fixed a twelfth, one a fifteenth, and another a seventeenth, of the span or chord of the arch, as the proper proportion for the depth of the stones at the apex. It is obvious enough, however, that although they are not false in themselves, because founded upon actual examples from executed structures, such general indications are not at all to be relied upon whenever the ordinary forms and dimensions are departed from; for they are given by their authors without their entering

into their reasons for doing so, or supporting them by arguments of any kind.

It is the same with regard to the rules given by Gauthier in his Treatise on Bridges; who distinguishes vaults accordingly as they are constructed either of hard or of soft stone; and to the former he allows a fifteenth of their span, when it exceeds 10 metres (32·8099 English feet); and to the others about 32 centimetres (12·5986 inches) more. Yet he takes no account of the different forms of curvature; and how defective his rule is, may be seen at once, when we find that the arches of the Pont de Neuilly support themselves with a depth of no more than 1·62 metres (5·315 feet) at their crown, whereas, according to Gauthier, it ought to be 2·6 metres (8·53 feet).

Boffrand has also drawn up tables for the same purpose, which give in general a greater ratio for the depth of the stones at the crown of an arch than Gauthier does, and consequently are no more deserving of attention.

In Perronet's work we meet with a rule which assigns for the depth of the arch-stones at the crown of an arch one twenty-fourth of its span; to which 325 millimetres (12·795 inches) are added, afterwards subtracting $\frac{1}{144}$ of the chord or span. This rule will be found to agree generally with the depths adopted in such bridges as have been built, especially in those whose arches are semicircular or segments of circles; yet some arches which have been executed appear to prove that the rule in question establishes too great a substance for the

structure, when the opening or span of the arch exceeds 30 metres (98·427 English feet).

We[1] are therefore of opinion that for a rule of this kind the following ought to be preferred: 1st, a depth of 0.33 metre (12·9923 inches) to all arches of less than two metres (6·5616 feet); 2ndly, for those from two metres (6·5616 feet) to sixteen metres (52·4928 feet) the depth should be $\frac{1}{48}$ of the span, increased by 0·33 metre (12·9923 inches); 3rdly, for those from 16 metres (52·4928 feet) to 32 metres (104·9856 feet), it ought to be $\frac{1}{24}$ of the span; 4thly, for arches exceeding 32 metres (104·9856 feet) the depth should be $\frac{1}{24}$ of the first 32 metres, and $\frac{1}{48}$ of whatever may be the overplus. The same rule will also serve both for semicircular and segmental arches, and to those of flatter and compounded arcs. There are many existing bridges of great size, the substance of whose arches is less than would be deduced from the foregoing rule.

In order duly to determine the substance of an arch, regard should be had both to the materials and to the mode of construction employed.

When it shall have been seen what are the effects which manifest themselves in arches after they are constructed, we shall be convinced that if an arch could be formed of incompressible materials, it could not settle, except in such degree as the resisting parts might not be sufficiently strong to support those which press upon

[1] '*We,*' in these Papers, must not be understood to mean the Editor, Editors, Author, or Authors of the work generally.

them; or, in other words, only in such degree as the abutments might not be sufficiently firm to resist the thrust of the arch.

Free-stone (*pierre de taille*) may be considered, for all practical purposes, as incompressible; and we find that if the arch-stones were to be placed one upon another without mortar, and without there being any other filling in or packing in such manner that there could be no bulging at all, the arch would support itself, provided the abutments were sufficiently firm, and the key-stone of such substance that it could not be crushed or splintered by the pressure it would have to sustain.

The depth of the key-stone would then at once be determined, if we knew the horizontal pressure the two halves of the arch exert upon each other, and the power of resistance of the stone they are constructed of; but it would be necessary to take into account the shaking from the motion of carriages to which such arches are exposed, in order not to give them an inadequate substance, more especially those of small diameter, upon which the pressure is less considerable.

If the pressure borne by the arch-stones in the crown of an arch of the boldest execution be calculated, we find that it is much less than what would be requisite to crush the stone. In the Pont de Neuilly, for instance, the horizontal pressure against the key-stone is about 14,000 kilogrammes (30,876·72 ℔s. av.) for each metre of its depth, supposing the masonry cut down to the level of the summit of the extrados; or about 185,000

kilogrammes (408,013·8 ℔s. av.), having reference to the weight of the filling in and of the pavement, and to that of carriages and passengers which it may have to sustain occasionally. The arches are built of Saillancourt stone; and according to M. Rondelet's experiments, a cubic piece of this stone of the first quality, whose side is 5 centimetres (1·9685 inches), requires a pressure of 3500 kilogrammes (7719·18 ℔s. av.) to crush it. Consequently, as the arch-stones at the summits of the arches are 1·624 metre (5·328 feet) deep, the force necessary to crush the stones, supposing it proportioned to the surfaces, would be 2,275,000 kilogrammes (5017467 ℔s. av.), that is, a pressure more than twelve times ~~greater~~ than what it is exposed to.

In the arch proposed to be built at Melun, with a span of 48·7 metres (157·4832 English feet), formed by the arc of a circle whose radius would be 65 metres (213·25 feet), the horizontal pressure against the key-stone would have been about 240,000 kilogrammes (529,315·2 ℔s. av.) for every metre of its depth, on account of the specific gravity of the stone (hard conglomerate sand-stone, *grès dur*), and of occasional additional weights. But if we take only 15,000 kilogrammes (33,082·2 ℔s. av.) as the resistance of this kind of stone on a surface of 25 square centimetres (3·875 square inches), which is below the results published by Rondelet, the force requisite to crush a stone 1·624 metre (5·328 feet) deep, would be 9,744,000 kilogrammes (21,490,197·12 ℔s. av.), which is equal to more than forty times the actual pressure.

After this it would appear that we might adopt for

arches a considerably less degree of depth in the arch-stones than what our most experienced constructors usually give them. But these calculations suppose that the arch-stones press equally on each other throughout, and that after the construction is completed there is no play or vibration, and no greater pressure on one part than on another. This hypothesis would not be very far from the truth if the joints of the arch-stones were perfectly executed; and if packing were avoided, adopting indeed the method employed by the ancients in their works of the same kind, we may conclude that bridges might be constructed possessing greater boldness and lightness than are possessed by the generality of those which have hitherto been executed by the moderns.

Nevertheless, both the change produced in the stones themselves by the effects of time, and the inevitable imperfection in the execution of the joints, would alone deter us from reducing the key-stones of bridges to the substances, which, according to such calculation, would appear to be sufficient, because the slightest splintering or fraying would sensibly diminish their resistance, and possibly endanger the arch. But the practice of packing the joints, and the liability to settling which results from it, and hence again of changes in the forms of arches, appear to be the principal causes of the great differences between the substance indicated as necessary by the strength of the stone, and that which is usually given.

In effect, the packing being, previously to the mortar becoming perfectly dry, the only intermediate substance

by which the arch-stones bear one upon another, it is evident that the resisting surface is not that of the entire surface or bed of the joint, but that it is reduced to that of the packing. Hence, it would seem possible to determine the depth of the arch-stone, in such manner that the superficies of the packing on which it rests were sufficiently large that the corresponding parts of the joint of the stone would be able to resist the pressure. But, it is necessary to remark, that the power of resistance of the stone cannot in this case be computed from experiments already known, as these have been made on small detached cubes.

It may also be remarked that the vertical settling of an arch is, *cæteris paribus*, greater in proportion as the depth of the key-stone is diminished. This settling depends, in part, on the shortening of the curve DE*d*, which connects the supporting points or extremities of the arch-stones, or those about which the stones have a tendency to turn in the settlement of the arch. This curve passes through the points of rupture D and *d*, and through E the summit of the extrados, and it is more or less depressed according as the key-stone is more or less deep. Hence we may easily perceive that, if the compression of the packing were known and the shortening the curve undergoes thereby, with regard to a determined pressure, we could compute the amount of settlement for an arch, corresponding to the given depth of its key-stone. The settlement would be greater in proportion as the curve DE*d* might be depressed: but the more considerable the settlement may

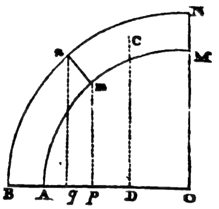
be, the more it may be found that it will be irregular, and that the form of the arch will become changed by means of it. This consideration induces us to endeavour to diminish the liability an arch may be under in this respect, by giving the key-stone more than depth enough to yield sufficient resistance to the greatest amount of pressure.

After what has been said it does not appear that it would be possible to establish any rule for determining the depth of the stones for the arches of bridges; besides, this question does not seem to admit of any general solution, because it evidently depends upon the nature of the materials, and upon the mode of construction employed. If the arch-stones were to be wrought according to the manner practised by the ancients, the beds being rubbed to an even surface, it would be a matter less arbitrary and uncertain. We might then reasonably presume that the arches would not settle at all when once constructed; and it would be sufficient to calculate the power of resistance of the stone, and its greater or lesser power of resisting the effects of time, and accidental injuries. But the mode usually practised does not admit of such calculation being applied, in the absence of experience, to all the elements of which it is composed. We must, therefore, content ourselves with consulting the examples with which the most skilful constructors have furnished us.

X.—ON THE PRESSURE TO WHICH THE STONE-WORK OF ARCHES IS EXPOSED.

It will be readily acknowledged that the question as to the degree of depth that ought to be allowed to the arch-stones of bridges is combined with a great variety of considerations. It is clear, however, that the greatness of the pressures to which key-stones are exposed, and the degree of resistance presented by the stone-work, form the principal elements of this inquiry, and that it is extremely necessary to pay attention to these points in the formation of large arches.

We shall express uniformly by Q the horizontal thrust of the arch, or the pressure which the two halves exercise on each other, and it will be seen, p. 15 (Theory), that this expression is determined by the condition to be the greatest possible force that must be applied horizontally in N, to prevent a portion mnNM of the vault to turn from top to bottom on the edge m. Let the co-ordinates AO, MO of the point M, be a and b, A being the origin, the length of the joint MN$=c$; and x and y the co-ordinates of the point m; the length of the joint $mn=z$; and the angle formed by this joint with the vertical, θ. The weight of the portion of the arch mnNM will be represented by G, as well as the parts of the construction which



it supports, and α will express the horizontal distance from the centre of gravity of this weight to the point A.

On these premises it will be clearly seen that by admitting the exact value of the horizontal thrust Q, calculated as above, we shall easily ascertain the perpendicular pressure that acts on any joint whatsoever mn. In fact, this pressure is nothing but the result of the forces Q and G, that influence the portion of the vault mnNM, decomposed perpendicularly at mn; that is to say, that it is expressed by

$$G \sin. \theta + Q \cos. \theta.$$

This expression for the vertical joint MN, is reduced to Q; and if the first joint AB is horizontal, it would be reduced for this joint to G, the quantity G being then the total weight of the half-arch.

On this occasion two remarks are necessary. The first, that it cannot generally be conformable to truth to calculate the horizontal thrust Q on the supposition of a force applied to the edge N; for the key-stones at the vertex of the arch press against each other on the whole elevation, or at least on a portion of the elevation of the plane of the joint. The pressure being thus subdivided over a certain space under the edge N, it acts, in order to prevent the descent of the portion of the vault mnMN, with the arm of a lever less than is supposed, when it is considered as being applied to N; and consequently, we find by this supposition, a value of Q

less than the real one. No sensible inconvenience however in general results from this discovery, as far as regards the equilibrium of the arch, but some may be felt with respect to the exact appreciation of the forces to which the stone-work is exposed.

The second remark is, that even when we arrive at an accurate knowledge of the value of the normal pressure operating on every joint, we cannot deduce from it the effort exerted on the stone-work, as we are ignorant of the manner in which this pressure is subdivided over the surface of the joint. Far from being able to admit that the pressure is equally spread over all this surface, it is known, on the contrary, that over all the arch, with the exception of a small number of joints, the pressure is principally exerted near one of the ends. This circumstance occurs chiefly at the vertex, where the pressure is felt near the upper end of the joint; on the joints of rupture placed in the haunches of the arch, where it acts near the lower ends; and finally, in the lower joints, under the springings, where the pressure manifests itself near the external arris. We must here suppose, conformably to general experience, that the lower parts of the arch have a tendency to be forced outwards.

The manner in which the pressure is spread over the surfaces of the joints is more uncertain, for this reason, that it depends on the precautions with which the beds of the arch-stones are wrought and disposed, on the distribution of the packing, the consistence of the mortar,

on the extent of the settlement of the arch, according to which the joints are more or less open, &c.

In order, however, to present some slight sketches on this subject, let us suppose an arch built of hard stone shall be wrought and set without mortar; suppose, moreover, that the lower part of the arch has scarcely substance and weight enough to resist the thrust, so that the equilibrium is nearly interrupted by the sinking of the upper, and the reversal of the lower parts. In that case, when we consider, in the first place, the joint MN, placed at the vertex of the vault, we may admit that if this joint were on the point of opening, the pressure at M would be nothing. We may suppose, moreover, that the stone admits of being compressed in a slight degree; this compression, which is almost imperceptible at the lower part of the joint, increases progressively from the point M to N, where it is at its maximum, and in the same way that the pressures experienced by the different portions of the surface of the joint increase also uniformly from the lower end M, where the pressure is nothing, to the upper end N, where it is in the greatest possible degree. Then still preserving the aforesaid denominations, let K be the value of the pressure at its maximum, which takes place at the upper end N, this value being referred to a unit of surface; the pressure exercised at the distance ν from the lower edge or arris M, will be $K\frac{\nu}{c}$, and the pressure exercised on the element $d\nu$ of the height of

the voussoir will be $K\frac{\nu d\nu}{c}$. It is besides to be observed, that the condition which determines the amount of the pressure exercised on all the elements ought to be such, that the sum of the moments taken with regard to the arris *m*, will be equal to the moment of the weight G of the portion of the arch *mn* NM, taken with regard to the same arris.

The equality of the sum of these moments may be thus expressed:

$$\frac{K}{c}\int_0^c d\nu.\ \nu(b-y+\nu)=G(\alpha-x);$$

or, by integration,

$$\frac{1}{6}K[3(b-y)c+2c^2]=G(\alpha-x).$$

$$\therefore K=\frac{6G(\alpha-x)}{3(b-y)c+2c^2}.$$

This formula will give, on the hypothesis which we have admitted, and which is not far from the truth, for an arch constructed as above, the value of the greatest pressure supported by the parts of the joint MN placed at the vertex of the arch. It is moreover evident, that as this value of K varies with the position of the joint *mn*, where the rupture is supposed to take place, it will be necessary to ascertain the position of this joint, which will give the greatest possible value for K.

The expression for the horizontal thrust, when cal-

culated on the supposition that it operates solely against the edge N, is

$$Q = \frac{G(\alpha - x)}{b - y + c}.$$

According to the data assumed above, the expression of this thrust is the sum of the elementary pressures $K\frac{\nu d\nu}{c}$, taken from 0 to c, that is to say, $\frac{1}{2}Kc$; or substituting the value of K we have,

$$\frac{3G(\alpha - x)}{3(b-y)+2c},$$

which is a value somewhat greater than the preceding.

With respect to the joint mn, which we shall suppose to be the joint of rupture, that is to say, the one which coincides with the greatest value of K, we shall be allowed equally to admit (since this joint is supposed to be ready to open), that the pressure is nothing at the upper edge n, and that it increases uniformly, setting out from this edge, till it reaches the lower edge m, where it is in the greatest possible force. If again we designate by K the maximum of the pressure with reference to a unit of surface, and by ν the distance of any point whatsoever of mn from the extremity n, we shall have $K\frac{\nu}{z}$ for the pressure which takes place in this point, as we have taken z to be the length of the joint. The sum of the pressures operating on the whole surface of the joint will then be

$$\frac{K}{z}\int_0^z d\nu.\nu, \text{ or } \frac{1}{2}Kz.$$

As we have given, at p. 61, an expression for the effort directed against the joint, by making it equal to that amount, we shall get

$$\frac{1}{2}\mathrm{K}z=\mathrm{G}\sin.\theta+\mathrm{Q}\cos.\theta,$$

$$\therefore \mathrm{K}=\frac{2(\mathrm{G}\sin.\theta+\mathrm{Q}\cos.\theta)}{z}.$$

This value of K, or the greatest pressure experienced by the parts of the joint of rupture *mn*, is precisely the double of the result obtained from supposing the effort equally divided over the whole surface of the joint.

The same considerations may be applied to the lower joint AB, which is ready to open in A, when the equilibrium of the vault is on the point of being interrupted. In order to calculate the value of K, which belongs to this joint, we may use the foregoing formula, in which G will represent the whole weight of the half-arch; and if the joint is horizontal, we shall have simply

$$\mathrm{K}=2\,\mathrm{G}.$$

Experience has fully proved that in arches similar to those usually adopted in modern bridges, not only the joints placed at the vertex, and at the points of rupture in the haunches, but also several joints near them, open when the equilibrium is once interrupted. In that case, the preceding formulæ are applicable to them. With respect to the intermediate joints, which have no tendency to open, it would not be more conformable to truth to suppose the pressure to be nothing at either of

the extremities of the joint. If we set out from the vertex of the arch, in going towards the haunches we pass the joints where the pressure tends to operate solely on the upper and outer arrises to those where it tends to operate solely on the lower and inner ones. We should find in the interval a joint in which the pressure does not tend to exert itself more on one end than on the other. This is determined by the condition that the direction of the one resulting from the horizontal pressure Q, and the weight of the portion of the half-arch that is above this joint, passes at an equal distance from the upper and lower ends. It may be supposed, with respect to the joint in question, that the pressure is equally spread throughout its whole depth. We should find, in like manner, a joint subject to the same condition, between the joint of rupture in the haunch, and the joint at the springing of the arch. It is obvious in this view, that if we apply to these intermediate joints the formula specified above, to determine the maximum value K of the pressure supported by the stone-work, we should have a value greater than the true one; so that these formulæ give, at least for the latter joints, the knowledge of a limit which the pressure cannot go beyond.

All the foregoing observations are, moreover, founded on the supposition that the parts of the arch are in a state of equilibrium which is near to being destroyed, so that the joints are almost in the act of opening at the points of rupture. But in reality, there will always be a surplus of resistance, for if the arch-stones may be supposed to have been well wrought and placed in contact, so as

to render bulging and change of form impossible, the joints will not open. In this state, it may be determined that there will be really no pressure upon those of the joints liable to rupture which are the least acted upon, as has been supposed above. We may therefore conclude that in calculating the value of K by the foregoing formulæ, we shall discover limits which the real pressure cannot pass, and from which it will be the more remote, as there will be a greater surplus of resistance in the lower parts of the arch.

If however we suppose the arch-stones to be set with packing and in mortar, or in other compressible materials which may admit of settlement and change of figure in the arch, and permit the joints to open or close, the hypotheses and the foregoing calculations will be at variance with the natural effects. It happens, in fact, that in consequence of the gaping of the joint at one of its extremities, and its binding at another, especially near the points of rupture, the arch-stones are entirely separated throughout a portion of their depth, and only bear upon one another at and near the end where the stress takes place. This effect was observed on striking the centres of the Bridge of Nemours, where the first arch-stones bore upon the impost only about $0^{m}\cdot32$ (1·05 feet) of the depth. The layer of mortar with which the joint had been filled before the centering was struck adhered to the bed of the impost, and was separated from that of the arch-stone on the remainder of the joint, the whole depth of which was $1^{m}\cdot30$ (4·265 feet).

It is difficult, in such cases, to arrive at an accurate

notion of the action to which the stone is subjected, and to establish a parallel between this action and that which it supports in experiments upon small cubes of the same material. Calculating upon the results of such experiments, it would appear that if the bearing were confined to the proportion above supposed of the surface of the bed of the arch-stone, there would be no occasion whatever for uneasiness as to the result.

It may be remarked also, that even if precautions are taken to adapt the joints to the probable results of settlements, by setting those open which are likely to bind in settling, and by setting the joints close which are likely to open, it does not follow that the pressure will so distribute itself as to produce the desired effect.[1]

It may be stated besides, that in diminishing the depth of the stones composing an arch, the resistance which the arch-stones oppose to pressure is not diminished in the same degree, especially if the diminution in that respect tends to increase the length of the radius of the intrados without lowering the vertex of the arch. In fact, by reducing the substance of the arch, the weight with which the upper part is charged is reduced, at the same time, in a considerable degree.

If the vertex of the curve of extrados is not depressed, then no change is made in the arm of the lever by which the pressures exercised against each other,

[1] The observations, of which the foregoing is the substance, have reference to a mode of practice which is not pursued in this country, and which, literally translated, would not be understood.

by the two halves of the arch, support the superincumbent weight; and even in the case of an arch which is the segment of a circle, or of one composed of various curves, this arm of the lever is increased, because the effect of a diminution of weight in the upper part of the arch is to depress the joints of rupture. Such being the fact, we need not be surprised to find a considerable difference between the degrees of thickness allowed by different architects to arches having the same dimensions, and again, to the diversity that is found with respect to the resistance which various sorts of stones offer to pressure. For though, in general, stone that offers the most resistance is likewise the heaviest, the difference found between the degrees of resistance of which various sorts of stone are capable, is much greater than that which exists between their specific gravities respectively; so that with harder stone the same solidity may be obtained with much less substance.

XI.—ON THE THEORY OF ARCHES, AND OBSERVATIONS ON THE FUNDAMENTAL PRINCIPLES ON WHICH IT IS GROUNDED.

In the works of Perronet we find a description of the effects which manifested themselves in the large arches which were constructed under his direction, both whilst the arch-stones rested upon the centres, after the keystones were set, and when the centering had been

struck. He has pointed out methods of striking the centering so as not to subject the curvature of the arches to alteration, and noticed what precautions are necessary to be taken in setting the voussoirs. All the remarks which he has published constitute a consistent system, and throw much light on this subject.

It may be generally remarked, he observes,[1] that the lower courses of arch-stones may be set without the assistance of centering, which becomes necessary only when they begin to slip upon one another, and this usually happens when the beds of the joints form with the horizon an angle of about 40 degrees. At and after that point, the centres begin to support a portion of the weight of the arch-stones, they spread at the base, and where a reversed centre is adopted, it rises at the vertex, unless hindered from doing so by the application of a weight more or less considerable.

The arch of St. Edmé, at Nogent-sur-Seine, appears to be the work for which these effects have been observed and described with the greatest care. Its form is a compound, with a span of $29^{m}\cdot24$ (95·9 feet), and a rise of $8^{m}\cdot77$ (28·77 feet), the depth of the key-stones being at the vertex $1^{m}\cdot62$ (5·31 feet). Each half of the arch is composed of 47 radiating courses of stones, without including the key-stone. The first twenty voussoirs being placed, the five last separated in consequence of

[1] Works of Perronet. Essay upon setting and striking the centres of bridges.

the settling of the centering on which they were placed; the joint opened 20 millimetres ('7874 inches) at the extrados above the 15th course, and there was a vertical disjunction between the arch-stones and the horizontal courses of the spandrels, the effect of which was felt at the seventh course. As the structure advanced these joints closed, and the point of separation between the acting and resisting points having been carried higher by the addition of a greater number of arch-stones, the joints opened at the extrados about 2 millimetres ('0787 inches) from the 26th to the 31st course.

In the bridge of Neuilly, the arches adjacent to the abutments, which are composed of fifty-six radiating courses on each side of the key-stone, the joints successively opened at the extrados, as the setting of the arch-stones advanced, from ½ millimetre ('0197 inches) to 5 ('1968 inches) and 7 millimetres ('2756 inches), from the 11th to the 36th course. Similar effects have been witnessed in several other bridges.

After the key-stones are placed, the effects produced by the weight of the arch-stones show themselves in a different manner. The centres, which before had been loaded in the lower part, giving the vertex a tendency to rise, are now loaded at the summit, and tend, in consequence, to rise in the haunches.

In the arches of the bridge of Neuilly it was observed that when the last joints which had opened at the extrados closed, new joints began to open at the intrados, commencing at the key-stone on both sides,

from ½ millimetre (·0197 inches) to 2 millimetres (·0787 inches), and extending from the 11th to the 36th course. In the bridge of Nogent, the vertical disjunction which had taken place between the arch-stones and the courses of the spandrels almost entirely disappeared, and the last joints that opened at the extrados, in the upper parts of the arches, also closed up.

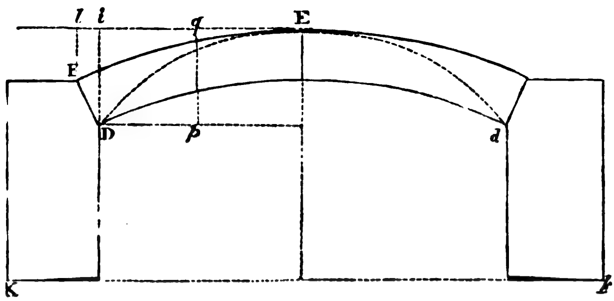
Before the centering of this last mentioned bridge was struck, three straight lines were drawn on the heads of the arch-stones, corresponding from the head of the 28th arch-stone on one side, to the head of that on the other; the other two were inclined and drawn over the haunches of the arch from the extremities of the first line to the points where the 7th course meets a tangent vertical to the springings of the arch. The positions of the extremities of these lines were considered to be fixed points, and the object was to ascertain, by means of the change that might be perceptible in their position and in their form, what would be the play or movement of the arch-stones during the settlement of the arch.

The curvature of the upper line indicated a vertical settlement, which went on uniformly diminishing from its middle to its extremities. As for the other two lines, they formed in their curvature a point of inflexion at the meeting of the joint of the 16th and 17th courses, which indicated, besides the vertical settlement, and the closing of the joints in the upper courses, as far as and comprising the 17th, a similar closing up in the joints of the inferior part, which was, besides, found to be thrust towards the abutments.

It is a natural conclusion, from these observations, which may be repeated on all constructions of the same kind, that the upper part has no tendency to drive back the lower parts by sliding upon the joints of rupture, as was supposed according to La Hire, and, consequently, that the results derived from calculations upon that hypothesis must be erroneous. To obtain an accurate idea of the nature of the thrust, it is necessary to consider successively the two principal epochs in the construction of an arch.

When most of the arch-stones are set, and are approaching towards the key-stone, the centre is loaded considerably, as it supports the whole weight of the arch, and it suffers in consequence an action which is especially felt towards the vertex. Every stone tends downward in proportion to its proximity to the key-stone, and it is clear that any stone in descending must turn on its lowest axis, whence the joint must open at the extrados. This separation is especially sensible at the point where, because of the inclination of the beds of the joint compared with the direction of the weight, the vertical action distributes itself more unequally over the subjacent arch-stones, and it is for this reason that, in the bridge of Nogent, the most open joint was found to be placed near the 26th course.

When the key-stone is set, and the centering struck, the upper parts of the arch DE and *d*E are no longer supported but by their mutual pressure, and, by reason of the settlement that takes place, their point of common support is of course carried to E at the



extrados: the joints then tend to close up, as has been constantly observed, and some architects have endeavoured to add to this effect by driving wedges, the object of which is to augment the solidity of the arch, at the same time that the energy of the pressure which these two parts of the arch direct against one another, and by means of which they are mutually supported, completes the compactness of the structure.

The effort of this pressure is however necessarily carried towards the abutments, and the lower parts of the arch, which it has a tendency to throw out by making them turn upon their exterior arrises K and *k*. Each half of the vault is separated into two parts, at certain points D and *d*, which serve as points of support to the upper parts, and by means of which their action is transmitted to the abutments; these points of support are necessarily found at the intrados. If the abutments have not stability enough to resist the action of the arch, the four parts will fall, turning upon the points K, D, E, *d* and *k*. If they have power to support it, the effect of the settlement will be confined to closing up the joints at the extrados near the point E, and at the in-

trados near the points *d* and D, and to make them open at the intrados near the point E, and at the extrados near the points D and *d*.

The position of the points *d* and D, which have been termed points of rupture, and which it is extremely important to know accurately, depends on the figure of the arch and the distribution of the weight which it supports. In the bridge of Nogent, of which we have spoken above, the position of the joints liable to rupture was naturally indicated, between the 16th and 17th courses of arch-stones, by the points of inflexion on the two lower lines drawn on the heads of the arch-stones. It was not possible to discover it by the same means in the bridge of Neuilly, by reason of the form of the heads of the arch-stones, which are in an arc of a circle, but it was found that the points of rupture were placed between the 26th and 27th courses, because it was in that spot that the joint opened most at the extrados.

It is obvious that arches can sink only in the same proportion that the stones nearest to the key-stone, to the points of rupture, and the base of the abutments, detach themselves one from another, by turning on their upper or lower arrises. The tenacity of mortar may oppose itself to this effect, and that tenacity may be thus sufficient to hold the arch together, as we sometimes find in ancient structures, when the piers or abutments have not the power of resistance which they should have. At the same time, the adhesive power of mortar must not be trusted to, because it does not take effect until after a lapse of time; and although by letting the centering

remain, we may afford the masonry time to become perfectly firm, it is not advisable to reckon upon any degree of strength to be acquired by that means, and which it would be difficult, besides, to compute with any degree of certainty.

The preceding remarks are fully confirmed by the observations which we have made on several arches that were in danger of falling, and on those which we have ourselves had occasion to take down. With respect to the latter, we took care to have horizontal trenches carefully formed in their piers, and we have uniformly observed that the first disjunctions appeared at the intrados near the key, that others were formed afterwards towards the haunches, where their greatest breadth was at the extrados, and that, in short, the upper part was depressed by dividing itself into two principal parts, each of which overturned the pier that it had stood upon.

This theory is equally in unison with the direct experiments which we undertook to make on this subject. We constructed arches in the full semicircle, and in the flat-vaulted arcs lowered $\frac{1}{3}$ and $\frac{1}{4}$, and others in segments of a circle. Their span was 65 centimetres (25·59 inches) ; the voussoirs, 27 millimetres (1·063 inches) in breadth at the inside, were made of wood, and cut with the utmost exactness. We endeavoured to destroy the equilibrium between the thrust of the upper parts, and the stability of the lower ones, both by diminishing the substance of the piers, and by loading the summit of the arch, and we have constantly remarked

that the rupture had a tendency to operate after the manner which we have described.

These experiments have been repeated on a larger scale by M. Boistard.[2] The arches which he employed were constructed with great exactness with voussoirs of bricks wrought as stones, the thickness and height of the joints of which were 109 millimetres (4·29 inches). The span of the arches was $2^{m}\cdot274$ (7·46 feet), and their length $0^{m}\cdot22$ (8·66 inches).

Arches varying in form from that of a semicircle to almost perfect flatness have been made with these materials, the versed sines of which were respectively $\frac{1}{4}$, $\frac{1}{9}$, and $\frac{1}{17}$ of the aperture. They were constructed with a centering, and their rupture occurred upon the centering being let down vertically; whether it was occasioned by loading the crown of the arch, or by diminishing the thickness and strength of the abutments.

Each of the arches which we have mentioned was subjected to three principal trials. In the first, the voussoirs were cut down to about 108 millimetres (4·25 inches) in depth, and, as this substance was not sufficient to enable them to support themselves when the centering was struck, a certain number of the voussoirs of the upper part sunk in a vertical direction with it, and made a lodgement on its summit. The two lower parts of the arch then produced the effect of two salient arcs, and divided themselves into two portions. The

[2] See a work entitled *Recueil d'expériences et d'observations faites par M. Boistard.*

last joints of each part opened at the intrados, near both the upper extremities, and the springings of the arch, and the rupture had a tendency to show itself towards the middle, where the voussoirs did not touch the centering, and where the joints opened at the extrados.

In the second trial, in which the voussoirs were again cut down, a certain number of them in the lower part were embraced by a cord that was drawn over the extrados, and which was kept stretched by a weight. The pressure which this cord produced over the last voussoirs counteracted the tendency to separate, which the upper parts of the arch produced; and it has been constantly observed—1st, that if the weights that produced the tension of the two cords were not sufficient for the equilibrium, the arch broke, opening at the intrados near the key-stone and near the springings, where the last voussoir tended to swing round upon its exterior arris, and at the extrados in the haunches; 2ndly, that in case the weights were sufficient to maintain the equilibrium, the same joints opened again in the same manner, in consequence of an unavoidable movement, but the action of the weights had a tendency to bring them together again, and they opened and shut alternately by a sort of oscillating motion, during which the parts of the arch turned alternately in the two directions round the points of support which the arrises of the consecutive arch-stones presented to them; 3rdly, that finally, when the tension of the cord was so great that the pressure it exercised on the lower parts of the

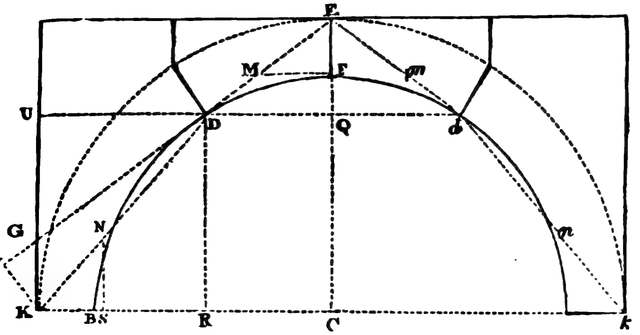
arch was capable of making the upper parts shoot up, the same effects were displayed in an opposite direction, that is to say, that the arch burst at the key, where it opened at the extrados, at the haunches, where it opened at the intrados, and at the springings, where the last voussoir turned round on its inner arris. When the arches were raised on piers, the effects were precisely the same, except that the piers acted with the lower parts of the arch, which, in falling, tended to turn round the outer arris of the base of the piers, instead of turning round that of the voussoir placed at the springings.

In the third trial, buttresses were raised, and the spandrels of the arch were filled up with masonry to a level with the vertex, where the arch was still 108 millimetres (4·25 inches) in depth. When the stability of the abutments was sufficient to resist the thrust, the arch retained its primitive figure after the centering was struck. When the weight on the upper part was increased, the arch broke in the usual way, and the position of the joints of rupture in the haunches was determined by the weight, and the manner in which it was distributed at the crown of the arch, and by the height of the piers on which this arch was sometimes raised. It was generally observed that when the arches were not raised on piers, the rupture had a tendency to appear at an angle of about 30 degrees of the semicircle describing the semi-arch, or towards the angle of 50 degrees of the small arc, in the flat-vaulted arch described with three arcs of a circle. The point of rupture usually

shoots upwards when the height of the piers is increased, and where the crown of the arch is overloaded.

All these experiments, which we regret our inability to give more in detail, completely confirm the principles which we have laid down as to the nature of the settlements of arches.[3]

The result is, that in investigating the principles of their equilibrium, we may (without any material error) consider them as systems of four levers KD, DE, E*d*, *d*K, each loaded with the respective weight of the parts



of the vault that correspond with it, and capable of turning round the points of support K, D, E, *d*, *k*, where they are connected by hinges, as it were: the position of the points *d* and D depends, for each parti-

[3] We find in the *Etudes relatives à l'art des constructions*, published by M. Bruyère, inspector-general of roads and bridges, first edition, the details of some experiments made about the year 1784 by M. Lecreulx, on the occasion of the construction of the bridge of Frouart. They lead to the same results as those specified above, and are particularly remarkable for ascertaining the effect of the division of a mass of abutments into many parts.

cular case, on the form of the arch, and the distribution of the weight with which it is loaded.

The same principle holds good with regard to arches in the form of the arc of a circle, and to plate-bandes or flat arches, where the vault and its abutments form a system precisely similar. But it is here necessary to remark, that, on account of the shape of the arch, the position of the points of rupture is situated at the springing, unless the vault happens to have the versed sine of the arc described, but small compared with the radius. The experiments made by M. Boistard prove that an arc whose versed sine is $\frac{1}{4}$ of the chord has its joints of rupture placed at the springings, where the haunches of the vault are filled up with masonry.

The calculation applied to the foregoing theory is clear of all difficulties. The points N, M, n, m, being those in which the levers are met by verticals which pass through the centres of gravity of the corresponding parts of the vault, we may suppose that these levers are loaded at those points with four weights equal to those of these parts, and it will be requisite to determine the relation which must exist between the weights and the direction of the levers, in order that the equilibrium may be maintained.

Let us only consider, for the sake of more simplicity, the half of the vault, which is divided into two symmetrical parts by the axis EC, and let us suppose, at first, that the point D is fixed; and let μ be the weight applied at M. No change will be made in this system by substituting two other weights; the one

applied at E, and represented by $\mu . \frac{FQ}{EQ}$,* and the other applied at D, and represented by $\mu . \frac{EF}{EQ}$. The point E will then be loaded by a weight equal to

$$2\mu . \frac{FQ}{EQ},$$

and there will result from it, in the direction of the lever ED, a pressure represented by

$$\mu . \frac{FQ}{EQ} . \frac{ED}{EQ},$$

and which, if the point D were really fixed, would be destroyed by its resistance, as well as the effect of the vertical force $\mu . \frac{EF}{EQ}$ which is applied at the same point. But as this point D is situated at the extremity of another lever, which has its point of support in K, and which is loaded at the point N with a new weight, which may be represented by ν, it will be necessary, in order to maintain the equilibrium, that the sum of the moments of these different forces, taken with reference to the point K, be nothing, which will give the equation

* Let W = weight at E, and ω the weight at D; then $W + \omega = \mu$,

$W+\omega : W :: DE : DM$

$W+\omega : \omega :: DE : EM$

but $DE : DM :: EQ : FQ$

and $DE : EM :: EQ : EF$

$\therefore W+\omega : W :: EQ : FQ$

$W+\omega : \omega :: EQ : EF$

$$W = \frac{(W+\omega) FQ}{EQ} = \frac{\mu . FQ}{EQ}$$

$$\omega = \frac{(W+\omega) EF}{EQ} = \frac{\mu . EF}{EQ}.$$

$$\mu.\frac{FQ}{EQ}.\frac{ED}{EQ}.KV=\mu.\frac{EF}{EQ}KR+\nu.KS,$$

KV being a perpendicular let fall from the point K on the line ED produced, KS and KR the horizontal distances from the point K to N and D.

It will then be seen that

$$KV=\frac{KU\cdot DQ-DU\cdot EQ}{ED},*$$

and substituting this value in the preceding equation, it becomes

$$\mu.\frac{FQ}{EQ}.\frac{DQ}{EQ}KU=\mu.KR+\nu.KS;$$

and if we wish the system to possess stability, we must have

$$\mu.\frac{FQ}{EQ}.\frac{DQ}{EQ}.KU.<\mu.KR+\nu.KS.$$

It is plain that the value of the thrust, and consequently the thickness that must be allowed to the piers, increase with the value of FQ, that is to say, when the centre of gravity of the upper parts of the vault approximates to its vertex. It is the same for the horizontal pressure which the two parts of the vault

* The triangles GKV, DGU and DEQ, are evidently similar

$$KV : KG :: DQ : ED$$

$$\therefore KV=\frac{DQ\cdot KG}{ED}=(KU-GU)\frac{DQ}{ED}.$$

$$=\frac{KU\cdot DQ-GU\cdot DQ}{ED},$$

but $DQ : EQ :: DU : GU$

$$DQ\cdot GU=DU\cdot EQ$$

$$\therefore KV=\frac{KU\cdot DQ-DU\cdot EQ}{ED}.$$

exert upon one another in the case of the equilibrium, as is clear from the form of its expression, which is evidently

$$\mu.\frac{FQ}{EQ}.\frac{DQ}{EQ}.$$

We agree with the remark of M. Boistard, that a mistake has hitherto prevailed by seeking to make the sinking of a vault depend on the diminution of the length of the curve of the intrados. It evidently depends on the shortening of a curve DEd (see figure, page 75), which joins the points by which the arch-stones lie on each other near the joints of rupture, and which may be considered also as joining the points of support of the intermediate arch-stones. The nature of this curve is not easily determined, but it may be considered, in its application, without any sensible error, as being of the same species as the curve of the intrados.

XII.—APPLICATION OF THE THEORY TO DETERMINE THE THICKNESS OF PIERS AND ABUTMENTS.

The equation $\mu\frac{FQ}{EQ}.\frac{DQ}{EQ}.\ KU=\mu.\ KR+\nu.\ KS$

contains all that is necessary to resolve the question which makes the subject of the present section, and the only point is to find a value of BK that will satisfy this equation, or rather which shall render the second member greater than the first, so that the arch may have sufficient stability. That, however, supposes that the position of the points of rupture of D and d is

known *à priori*, a circumstance which does not generally occur, but which it is necessary previously to determine.

In order to do so, we must observe that the points should be so placed, that the momentum of the force that tends to overturn the lower part should be the greatest possible in proportion to the forces that tend to retain it in its position. It is necessary therefore to seek for a value of the arc BD that corresponds with the *maximum* of the expression

$$\frac{\mu . \frac{FQ}{EQ} . \frac{DQ}{EQ} . KU}{\mu . KR + \nu . KS}.$$

The calculation is nearly impracticable for arches forming a complete semicircle by reason of the transcendental qualities which the nature of the circle introduces into this formula, and it becomes entirely so for arches that are composed of several arcs. We must, therefore, have recourse to an indirect method, which consists in assuming different positions of the point D, and determining for each the corresponding value of BK. We may be guided in this species of guess-work by the example of well known bridges, the form of which resembles that of the one in question. It is clear, moreover, that the greatest value assigned to BK will be one that must be taken into consideration, and that the position of the points of rupture will be determined by the corresponding value of the arc BD.

The following table contains the results of this calculation for the vaults that are most frequently in use. We must suppose their span to be 20^m, (=65·618 feet),

the thickness at the key to be one metre, (=3·2809 feet), and the extrados level.

Nature of the arches.	Thickness of the abutments.	Position of the points of rupture.
	Metres. Feet.	Degrees.
Semicircular	0·45 = 1·476	27
Flat arches $v.s = \frac{1}{3}$ span	0·66 = 2·165	45
Do. Do. $\frac{1}{4}$ do.	0·82 = 2·690	54
Circular arc of 60 degrees on piers 5 metres (= 16·4 feet) high	2·95 = 9·679	0

Note. The numbers of the degrees shown in the third column are counted from the abutments, and on the small arc in the flat arches, supposing them to be described with three arcs each equal to the sixth of the circumference.

The results contained in the above table are much under the ordinary dimensions, and the theory which we have explained points out degrees of thickness less considerable than those hitherto allowed to the piers of bridges.

On this subject it is proper to observe that the foregoing calculations suppose that the different portions of the vaults form solid masses, all the parts of which are perfectly connected, and not subject to any sinking. They suppose, likewise, that the abutments are built upon a foundation entirely incompressible, and that, in the fall of the arch, these abutments would turn, without any disjunction, round their exterior edge. These suppositions are generally far from truth. The fall of a bridge cannot occur without some disjunctions in the abutments, with whatever care they may have been constructed; and even should none occur, the abutments could not turn round their exterior edge, where the stones would necessarily be crushed under the pressure

they would have to support, which pressure it should be our business, for this reason, to subdivide over a sufficiently extensive surface. With respect to the incompressibility of the foundations, this condition is very difficult to be accurately ascertained, especially when the masonry is not placed on a platform resting on piles; and it is clear that the downfall of the greatest number of bridges ought to be attributed to the failure of the supports. But as the foundation is the more incompressible as the pressure it supports is distributed over a wider surface, the architect is compelled to increase the dimensions of the points of support in order to adapt the physical circumstances to the analytical hypothesis.[1]

[1] It appears that the experiments of Boistard gave rise to this new theory here treated of by Gauthey, and which Colonel Audoy has shown to coincide exactly with that of Coulomb, as far as rotation is concerned. The theory of this celebrated man is therefore more general, as it takes into account the tendency the voussoirs have to slide on each other; and, besides, it has the advantage of being simpler in its form. See Garidel's Tables des Poussées des Voûtes en Plein Cintre, and Navier's Notes on Gauthey.

THEORETICAL AND PRACTICAL PAPERS ON BRIDGES.

I.—ON THE THEORY OF THE ARCH.

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GENERAL CONDITIONS OF THE EQUILIBRIUM OF A STRUCTURE OF UNCEMENTED STONES.

A STRUCTURE may yield under the pressures to which it is subjected either by the slipping of certain of its surfaces of contact upon one another, or by their turning over upon the edges of one another; and these two conditions involve the whole question of its stability.

Let a structure MNLK, fig. 1, composed of a single row of uncemented stones of any forms, and placed under any given circumstances of pressure, be conceived to be *intersected* by any geometrical surface 1 2, and let the *resultant* aA of all the forces which act upon one of the parts MN21, into which this intersecting surface divides the structure, be imagined to be taken.

Conceive then this intersecting surface to change its form and position so as to coincide in succession with all the common surfaces of contact 3 4, 5 6, 7 8, 9 10, of the stones which compose the structure; and let bB, cC, dD, eE, be the resultants, similarly taken with aA, which correspond to these several planes of intersection.

In each such position of the intersecting surface, the resultant spoken of having its direction produced, will intersect that surface either *within* the mass of the structure, or, when that surface is imagined to be produced, *without* it. If it intersect it *without* the mass of the structure, then the *whole* pressure upon one of the parts, acting in the direction of this resultant, will cause that part to turn over upon the edge of its common surface of contact with the other part; if it intersect it *within* the mass of the structure, it will not.

Thus, for instance, if the direction of the resultant of the forces acting upon the part N M 1 2 had been $a'A'$, not intersecting the surface of contact 1 2 *within* the mass of the structure, but when imagined to be produced beyond it to a'; then the whole pressure upon this part acting in $a'A'$ would have caused it to turn upon the edge 2 of the surface of contact 1 2; and similarly if the resultant had been in $a''A''$, then it would have caused the mass to revolve upon the edge 1. The resultant having the direction aA, the mass will not be made to revolve on either edge of the surface of contact 1 2.

Thus the condition that no two parts of the mass should be made by the insistent pressures to turn over upon their common surfaces of contact is involved in this other, that the direction of the resultant, taken in respect to every position of the intersecting surface, shall intersect that surface actually *within* the mass of the structure.

If the intersecting surface be imagined to take up an infinite number of different positions, 1 2, 3 4, 5 6, &c., and the intersections with it, *a*, *b*, *c*, *d*, &c., of the directions of all the corresponding resultants be found, then the curve line *a b c d e f* joining these points of intersection is that to which I have given the name of the LINE OF RESISTANCE.

This line can be completely determined by the methods of analysis in respect to a structure of any given geometrical form having its parts in contact by surfaces also of given geometrical forms. And conversely, the form of this line being assumed, and the direction which it shall have through any proposed structure, the geometrical form of that structure may be determined, subject to these conditions; or lastly, certain conditions being assumed both as it regards the form of the structure and its line of resistance, all that is necessary to the existence of these *assumed* conditions may be found. Let the structure ABDC, fig. 2, have for its line of resistance the line PQ. Now, it is clear that if this line cut the surface MN of any section of the mass in a point *n* without the surface of the mass, then the resultant of the pressures upon the mass CMN will

act through *n*, and cause this portion of the mass to revolve about the nearest point N of the intersection of the surface of section MN with the surface of the structure.

Thus, then, it is a condition of the equilibrium that *the line of resistance shall intersect the common surface of contact of each two contiguous portions of the structure, actually within the mass of the structure;* or, in other words, that it shall actually go through each joint of the structure, avoiding none: this condition being necessary, that no two portions of the structure may revolve on the edges of their common surface of contact.

But besides the condition that no two parts of the structure should turn upon the edges of their common surfaces of contact, which condition is involved in the determination of the LINE OF RESISTANCE, there is a *second* condition necessary to the stability of the structure. Its surfaces of contact must no where slip upon one another. That this condition may obtain, the resultant corresponding to each surface of contact must have its *direction* within certain limits.[1] These limits

[1] The resistance of surfaces is *not exerted exclusively* in the direction of the normal, according to *an hypothesis,* which was probably introduced into the theory of Statics in order to simplify the investigations of those who *originated* that science, but which there seems no reason for retaining any longer. It is exerted in an infinite number of different directions included within a certain angle to the normal, or rather within the surface of a certain right cone, having the normal for its axis and the point of resistance for its vertex. *Any force,* however great, applied within this conical surface, will be sustained by the

are defined by the surface of a right cone, having the normal to the common surface of contact at the above-mentioned point of intersection of the resultant for its axis, and having for its vertical angle twice that whose tangent is the coefficient of friction of the surfaces. If the direction of the resultant be *within* this cone, the surfaces of contact will not slip upon one another; if it be without it, they will.

Thus then the *directions* of the consecutive resultants in respect to the normal to the point where each intersects its corresponding surface of contact, are to be considered as important elements of the theory.

Let then a line, ABCDE, fig. 1, be taken, which is the locus of the consecutive intersections of the resultants aA, bB, cC, dD, &c. This line I have called the LINE OF PRESSURE. Its geometrical form may be

resistance of the surface of the mass—and *no force*, however small, without it.

Let R represent a single force, or the resultant of any number of forces applied to a fixed surface, and let R' and R'' be the resolved parts of R in the directions perpendicular and parallel to the surface. Also let ρ be the inclination of R to the vertical, and f the coefficient of friction. The friction of the surfaces in contact is therefore represented by fR', and motion will, or will not, ensue according as R'' *is* greater or is *not* greater than fR'. Or, according as $\frac{R''}{R'}$ *is* greater or is *not* greater than f. Or, if $f=\tan. \phi$, according as $\tan. \rho$ is, or is not, greater than $\tan. \phi$, or as ρ is greater or is not greater than ϕ. In the remainder of this paper the angle ϕ, or $\tan.^{-1} f$, will be called *the limiting angle of resistance*. This principle of the resistance of a surface was, I believe, first given in my paper on the Equilibrium of the Arch, read before the Cambridge Philosophical Society in December, 1833, and published in the fifth volume of their Transactions.

determined under the same circumstances as that of the line of resistance. A straight line cC drawn from the point c, where the LINE OF RESISTANCE $abcd$ intersects any joint 5 6 of the structure, so as to touch the LINE OF PRESSURE ABCD, will determine the *direction* of the resultant pressure upon that joint: if it lie within the cone spoken of, the structure will not slip upon that joint; if it lie without it, it will.

Thus the whole theory of the equilibrium of any structure is involved in the determination with respect to that structure of these two lines—the line of resistance, and the line of pressure. One of these lines, the line of resistance, determining the *point* of application of the resultant of the pressures upon each of the surfaces of contact of the system; and the other, the line of pressure, the *direction* of that resultant.

The determination of both, I have shown, to be, under their most general forms, within the resources of analysis, and I have given general equations for their determination in that case, in which all the surfaces of contact or joints are planes—the only case which offers itself as a *practical* case. The analytical discussion of these equations is reserved for a subsequent paper; my present object is to give a general account of their application to various practical questions of construction. My researches have been confined to that class of prismatic structures, the geometrical form of any of which may be conceived to be generated by the motion, parallel to itself, of a vertical *plane* of given form and dimensions. Such structures are represented in figures 3, 4, 5, 6, &c.; they

include every form of the pier, the buttress, the plate bande, the revetment wall, the stone embankment, and the arch. I have supposed their common surfaces of contact to be planes, and the forces acting on them to be the weights of their parts, and other forces applied from without, but all acting in vertical planes, and applied uniformly along the horizontal breadth of each structure. I have first considered that class of structures (including all those enumerated above, except the arch) in which the planes of section are all parallel to one another. I have determined the general equations to the LINES OF RESISTANCE AND PRESSURE in respect to this great class of structures, and I have shown that when the planes of section are vertical planes, these two lines coincide.

In applying these equations I have first supposed the various external pressures sustained by each structure to be applied only to certain portions of its surface, as for instance its extremities, or the surfaces of a few of the stones near its extremities, and I have thence determined the form and direction of the line of resistance through the rest of the structure. Upon this hypothesis I find the equation to the line of resistance PRST, of the trapezoidal structure, fig. 6, to be (under its most general form) one of three dimensions; showing that if the mass be imagined to be sufficiently extended it will, under certain circumstances, cut the extrados in three distinct points, R, S, T; at each of which points rupture will therefore take place, unless some external resistance be there applied to the extrados.

The highest of these points, R, marks that section which must terminate the structure, that it may stand unsupported under its insistent pressures.

Certain assumed values of the arbitrary constant, which enter into the equation to this line of resistance, convert it into the line of resistance of a buttress, fig. 7, with one or both of its faces AB and DC inclined to the vertical: an inclined pier, figs. 8, 9; an upright pier, fig. 10; or a plate bande in a horizontal or an inclined position, and of an uniform and variable width, figs. 11, 12, 13.

In a pier or wall, figs. 7, 9, 10, of uniform thickness, whose face is inclined to the vertical at any angle, and which has its parallel planes of section inclined to it at any given angle, the line of resistance is always an hyperbola whose magnitude and position are readily determinable in terms of the thickness of the mass and the magnitudes, directions, and points of application of the insistent forces.

In the case of an *upright* pier or uniform wall, fig. 10, the asymptote of this hyperbola is a *vertical* line KE. The pier will not be overthrown by the insistent pressures, however high it may be built, provided this asymptote (determined in respect to those pressures) be found to be (as in the figure) *within* the mass of the pier.

In the case of the uniform plate bande, figs. 12, 13, where the planes of section are all vertical, the line of resistance is always a parabola, and it coincides with the line of pressure. The intersecting planes have, up to this period of the investigation, been supposed to be

parallel to one another. Let this hypothesis now be discarded, and, as the simplest case of a section of variable inclination, let its plane be supposed always to pass through the same horizontal axis. This case includes that of the circular arch under its most general form, and to this case my further researches have been limited.

I have supposed certain forces to be applied to one extremity of a structure thus intersected, and resting by its other extremity upon an immoveable base. As for instance a semi-arch, fig. 14, resting by its extremity B upon its abutments, and supported by a given force P, applied to the key-stone AD, instead of the pressure of an opposite semi-arch. On this hypothesis the equation to the line of resistance may be completely determined in respect to an arch of equal voussoirs subjected to any variety of loading. With a view to this general determination I have first supposed the loading to be collected over a single point X of the semi-arch; and on this hypothesis I have found the equation to the line of pressure in terms, of the inclination of the joint AD of the key-stone (that is, of the line CD) to the vertical, the angle ACB of the segment of the arch, the common depth AD of the voussoirs the point of application, and the magnitude of the force P and the weight X. This determination evidently includes the cases of the loaded Gothic and segmental arches; and were the magnitude and point of application of the force P known, it would constitute a complete determination of the equilibrium of the structure.

But unfortunately, in the actual case of the arch, this pressure upon the key is an unknown thing. We neither know its point of application nor its amount.

It is the pressure of the opposite semi-arch, or rather it is the resultant of an infinity of pressures exerted by the opposite semi-arch upon an infinity of points, by which that semi-arch is in contact with the face AD of the key; and the amount of this resultant, and whether it pass through the middle of the key-stone or its extremities, are necessary, but, up to this period of the investigation, *unknown* elements of the theory. Some other principle of mechanical action manifestly enters into the conditions of the equilibrium, and claims a place at this period of the discussion.

That other principle is this, that of all the pressures which can be applied to the key, different in their points of application and amount, but all consistent with the equilibrium of the semi-arch, that which it actually sustains by the pressure of the opposite semi-arch is the *least*. This condition of minimum pressure at the key supplies mathematically all that is required for the complete determination of that pressure, and perfects the theory.

The demonstration of it is easy. The pressure which an opposite semi-arch would produce upon the side AC of the key-stone, fig. 2, is equal to the tendency of that semi-arch to revolve forwards upon the inferior edges of one or more of its voussoirs. Now this tendency to motion is evidently equal to the least force which would support this opposite semi-arch; supposing the semi-

arches, therefore, to be equal in every respect, and equally loaded, it is equal to the least force which would support the semi-arch ABDC.

Suppose the mass ABDC, fig. 2, to be acted upon by any number of forces among which is the force Q being the resultant of certain resistances, supplied by different points in a surface BD, common to the intersected mass and to an immoveable obstacle BE.

Now it is clear that under these circumstances we may vary the force P, both as to its amount, direction, and point of application, without disturbing the equilibrium, provided only the form and direction of the line of resistance continue to satisfy the conditions imposed by the equilibrium of the system.

These have been shown to be the following,—that it no where *cut* the surface of the mass, except at P, and within the space BD, and that it no where cut any section MN of the mass, or the common surface BD of the mass and obstacle, at an angle with the perpendicular to that surface, greater than the limiting angle of resistance.

Thus, varying the force P, we may destroy the equilibrium, either, first, by causing the line of resistance to take a direction without the limits prescribed by the resistance of any section MN through which it passes, that is, without the cone of resistance at the point where it intersects that surface; or, secondly, by causing the point Q to fall *without* the surface BD, in which case *no resistance* can be opposed to the

resultant force acting in that point; or, thirdly, the point Q lying within the surface BD, we may destroy the equilibrium by causing the line of resistance to cut the surface of the mass somewhere between that point and P.

Let us suppose the limits of the variation of P within which the first two conditions are satisfied, to be known; and varying it, within those limits, let us consider what may be its *least* and *greatest* values so as to satisfy the third condition.

Let P act at a given point in AC and in a given direction. It is evident that by diminishing it under these circumstances, the line of resistance will be made continually to assume more nearly that direction which it would have, if P were entirely removed.

Provided then, that if P *were* thus removed, the line of resistance would cut the surface, that is, provided the force P be necessary to the equilibrium; it follows that by diminishing it, we may vary the direction and curvature of the line of resistance until we at length make it *touch* some point or other in the surface of the mass.

And this is the limit; for if the diminution be carried further, it will *cut* the surface, and the equilibrium will be destroyed. It appears then that under the circumstances supposed, when P, acting at a given point and in a given direction, is the least possible, the line of resistance *touches the interior surface or intrados of the mass*.

In the same manner it may be shown, that when it is the greatest possible, the line of pressure touches the exterior surface or extrados of the mass.

I have here supposed the direction and point of application of P in AC to be given; but by varying this direction and point of application, the contact of the line of resistance with the intrados of the arch may be made to take place in an infinite variety of different points, and each such variety supplies a new value of P. Among these, therefore, it remains to seek the *absolute* maximum and minimum values of that force.

In respect to the direction of the force P, or its inclination to AC, it is at once apparent that the least value of that force is obtained, whatever be its point of application, when it is *perpendicular* to AC.

There remain then two conditions to which P is to be subjected, and which involve its condition of a minimum. The *first* is, that its amount shall be such as will give to the line of resistance a point of contact with the intrados. The *second*, that its point of application in the key-stone AC shall be such as to give it the least value which it can receive, subject to the first condition.

I have determined the value of P subject to these conditions in a paper read before the Cambridge Philosophical Society in May 1837, and published in the 6th volume of their Transactions. The equations involving that value admit of a complete solution, and determine it for every form and dimension of the broken

or Gothic arch, and the complete segment, and for every circumstance of its loading.

The condition however that the resultant pressure upon the key-stone is subject in respect to the *position* of its point of application on the key-stone to the condition of a minimum, is dependent upon hypothetical qualities of the masonry. It supposes an unyielding material for the arch-stones, and a mathematical adjustment of their surfaces. These have no existence in practice. On the striking of the centres the arch invariably sinks at the crown, its voussoirs there slightly opening at their lower edges, and pressing upon one another exclusively by their upper edges. Practically the line of resistance then, in an arch of *uncemented* stones, *touches the extrados* at the crown; so that only the first of the two conditions of the minimum stated above actually obtains: that, namely, which gives to the line of resistance a contact with the intrados of the arch. This condition being assumed, all consideration of the yielding quality of the material of the arch and its abutments is *eliminated.* It will thus be discussed in what remains of this paper.

To simplify the analytical discussion of the question I have hitherto assumed the load upon the semi-arch to be placed over a single point of it X, fig. 14. I now imagine it to be distributed in any way over the extrados, but symmetrically in respect to the two opposite semi-arches. The centre of gravity of this load on each semi-arch being determined, it is evident that the *horizontal thrust* P on the key-stone of the arch will be the same

if the whole load upon it be imagined to be collected in these two centres of gravity. I determine then the horizontal thrust P on this hypothesis of a concentrated loading: this determination being made, the data necessary to the analytical discussion of the question are *complete*, all the forces acting upon a mass ASTD of the arch and its loading intercepted between the crown and any inclined position CT of the radius are given, and the equation to the true line of resistance under any given circumstances of loading is determinable in terms of the radius vector CR and the angle ACS. The equation determining the value of P is unfortunately one of a high order, involving circular functions of complicated forms; and the solution of it otherwise than by approximation is perhaps to be despaired of. The small value of the ratio of the depth AD of the voussoirs, in the majority of practical cases, to the radius CA of the arch in terms of which ratio the value of P is expressed, suggests a developement of the value of P in a series of terms ascending by powers of this ratio. To effect this developement I have called to my aid the theorem of Lagrange, using two terms only of that theorem, and not therefore extending the approximation beyond the first power of the ratio. It might perhaps be expedient in some cases to extend it to the second; beyond this limit no practical enquiry need however be carried.

The line of resistance being fully determined, the *point* Q, fig. 2, where the resultant pressure of the whole semi-arch intersects, the supporting surface BD

of the abutment becomes known, and also the *direction* of this resultant pressure. Now all the circumstances which determine the equilibrium of an abutment, subject at a given point to a given insistent pressure, I have before discussed, and I have determined its line of resistance under these circumstances: that line of resistance evidently unites with that of the arch at this point—this line of pressure is therefore completely known, and the conditions of the equilibrium of the piers or other abutments of the arch, and of the arch itself, are determined.

I have hitherto considered the form of the solid to be given, together with the positions of the different sections made through it, and I have thence the forms of its lines of resistance and pressure, and their directions through its mass.

It is manifest that the converse of this operation is possible.

Having given the form and position of the line of resistance or of pressure, and the positions of the different sections to be made through the mass, I may for instance enquire what form these conditions impose upon the surface which bounds it.

Or I may make the direction of the line of resistance or pressure and the form of the bounding surface subject to certain conditions not absolutely determining either.

For instance, if the form of the intrados of an *arch* be given, and the direction of the intersecting plane be always perpendicular to it, and if I suppose the line

of pressure to intersect this plane always at the same given angle with the perpendicular to it, so that the tendency of the pressure to thrust each from its place may be the same,—I may determine what under these circumstances must be the extrados of the arch.

If this angle *equal* constantly the limiting angle of resistance, the arch is in a state bordering upon motion, each voussoir being upon the point of slipping downwards or upwards, according as the constant angle is measured above or below the perpendicular to the surface of the voussoir.

The systems of voussoirs which satisfy these two conditions are the greatest and least possible.

If the constant angle be zero, the line of pressure being every where perpendicular to the joints of the voussoirs, the arch would stand even if there were no friction of their surfaces.

It is then technically said to be equilibrated, and the equilibrium of the arch according to this single condition constituted the theory of the arch so long in vogue, and so well known from the works of Emerson, Hutton, Whewell, &c. It is impossible to conceive any arrangement of the parts of an arch by which its stability can be more effectually secured, *so far as the tendency of its voussoirs to slide upon one another is concerned:* there is however, I believe, no practical case in which this tendency really affects the equilibrium. So great is the *limiting angle of resistance* in respect to all the kinds of stone used in the construction of arches that

it would be exceedingly *difficult* to construct an arch, the resultant pressure upon any of the joints of which should lie *without* this angle, or which should yield by the *slipping* of any of its voussoirs.

The theory stated above readily explains the phenomena observed in the settlement of the arch.

In the case of a trapezoidal mass placed in any inclined position and intersected by parallel planes, I have investigated in my first paper (Cambridge Phil. Trans., vol. v. part 3) the equation to the line of pressure, and I have ascertained it to be one of three dimensions, having a point of contrary flexure. This case of a trapezoidal mass thus obliquely placed approaches sufficiently near to that of the arch, to indicate the existence of a similar point of contrary flexure in the line of pressure to the arch. Now it is evident that since the directions of the pressures on the successive joints are all tangents to the line of pressure, this point of contrary flexure in it, shows a change to take place, somewhere, in the directions of the pressures in respect to their inclinations to the joints; the inclinations of the pressures at all the joints above a certain one, being downwards or *towards* the intrados, and those below it upwards or *from* the intrados.

Hence, therefore, it appears that the tendency of the pressure is to cause all the voussoirs above the joint spoken of to slide *downwards*, and those beneath that joint, *upwards*; and that these effects may be expected to follow the striking of the centre of the arch ; the

weight being then suddenly thrown upon the voussoirs, and these admitting of a certain degree of motion, in the directions of the forces impressed upon them.

Now this is precisely what was observed at the bridge of Nogent, of the construction of which Perronet has left a detailed account.

Three straight lines were drawn upon the face of the arch before the striking of the centre, one of them stretching horizontally above the crown, and the two others lying obliquely from the extremities of this, towards the springings of the arch.

After the centre had been struck, the lines were observed to have assumed certain curved forms, indicating, in accordance with the theory, a downward motion in all the voussoirs above a certain point on each side, and an upward motion in the voussoirs beneath that point.

These observations have been confirmed by numerous others, and especially by those (made also by Perronet) at the Pont de Neuilly.

Let ABB′, fig. 15, represent an arch having the joints of its voussoirs perpendicular to the intrados as they usually are made. Let RQPQ′R′ be the line of resistance touching the intrados at Q and Q′, and the extrados at the crown in A. The material of the arch may therefore be expected to yield more particularly about the points A, Q and Q′, than any other; a greater proportion of the pressure being there thrown upon the *edges* of the voussoirs.

If by reason of such yielding, or from any other alter-

ation in the forces impressed upon the mass, or in the circumstances of their application, the form of the line of resistance be altered, it may manifestly be expected to intersect the surface of the mass first about *those points*; the least possible alteration of form being there sufficient to produce the intersection. And this being the case, the portion of the arch above Q and Q′ must separate into *two* portions, revolving at those points about the lower portions of the arch (see fig. 16) and at A, upon the extremities of one another.

Nevertheless this revolution is manifestly impossible unless the points Q and Q′ yield outwards. And this can only take place by the yielding of the material at Q and Q′, by the slipping back of the voussoirs there, or by the portions of the arch or its abutments beneath those points revolving outwards, in consequence of the intersection of the extrados by the extremities QR and Q′R′ of the line of resistance (fig. 15).

The last is in point of fact the cause which leads, in the great majority of cases, to the fall of the arch.

The extremity R of the line of resistance is made to cut the extrados of the arch, or the outer surface of the pier, by the diminution or removal of some force which acted there in opposition to the tendency of the arch to spread itself, and which kept the direction of the line of resistance within its mass,—the resistance of a mass of earth for instance, or the opposite thrust of some other arch springing from the same pier or abutment.

On the whole, then, it appears that in the commencement of its fall the arch will, according to this

theory, divide itself into six distinct portions, of which four will revolve about the points S, S′, Q, Q′ and A, as represented in the figure 16. Now this is what is uniformly observed to take place in the fall of the arch.

Gauthey, having occasion to destroy a bridge, caused one of its arches to be isolated from the rest ; and, the adhesion of the cement being sufficient to counteract the tendency of the pressure to rupture the piers, he caused them to be cut across. The whole then at once fell, the falling portion separating itself into four parts. Having constructed small arches of soft stone, and without cement, he loaded them until they fell. Their fall was always observed to be attended with the same circumstances. Before the arch finally yielded the stone also was observed to chip at the intrados about the points Q and Q′, round which the upper portions of it finally revolved.

Some experiments made by Professor Robison with *chalk* models were attended with slightly different results. Having loaded them at the crown until they fell, he observed first, that the points where the material began to yield were not precisely those where the rupture finally took place.

This fact presents a remarkable confirmation of the theory expounded in this paper.

It is manifest, that according to that theory, with any variation in the least force P, which would support the semi-arch if applied at its crown, there will be a corresponding change in the position of the point Q.

Now as the load upon the crown is increased, this

least force P is manifestly increased. The result is a corresponding variation in the form of the line of resistance, tending to carry its point of contact with the intrados lower down upon the arch.

This is precisely what Professor Robison observed. The arch began to chip at a point about half way between the crown and the point where the rupture finally took place.

The existence of the points Q and Q′, about which the two upper portions of the arch have a tendency to turn, and about which the material is first observed to yield, has long been known to practical men. The French engineers have named these points the points of rupture of the arch; and the determination of their position by a *tentative method* forms an important feature in the theory which they have applied to this important branch of Statics.

The theory of the equilibrium of the groin and that of the dome are precisely analogous to the theory of the arch.

In the former a mass springs from a small abutment, spreading itself out symmetrically with regard to a vertical plane passing through the centre of its abutment. The groin is in fact nothing more than an arch, whose voussoirs vary as well in breadth as in depth. The centres of gravity of the different elementary voussoirs of this mass lie all in its plane of symmetry. Its line of resistance is therefore in that plane, and its theory is embraced in that which has been already laid down.

Four groins commonly spring from one abutment;

each *opposite* pair being addossed, and each *adjacent* pair uniting their margins. They thus lend one another mutual support, partake in the properties of a dome, and form a continued covering.

The groined arch is of all arches the most stable; and could materials be found of sufficient strength to form its abutments and the parts about its springing, I am inclined to think that it might be safely built of any required degree of flatness, and that spaces of enormous dimensions might readily be covered by it.

It is remarkable that modern builders, whilst they have erected the common arch on a scale of magnitude nearly approaching perhaps the limits to which it can be safely carried, have been remarkably timid in the use of the groin.

I shall terminate this paper by a comparison of the theory developed in it with that of Coulomb, which was unknown to me until my researches had long been completed, and which, after an oblivion of more than sixty years among the pages of the Mémoires des Savants Etrangers, undisturbed by the many and fierce discussions to which the question has been subjected since that time in this country and elsewhere, has recently been exhumed and made the subject of valuable researches by M.M. Navier, Lamé and Clapeyron, and Garidel.

COMPARISON OF THE PRECEDING THEORY WITH THAT OF COULOMB.

According to the theory of Coulomb (Mémoires des Savants Etrangers, 1773,) as developed by the researches of Navier, (Résumé des leçons données a l'école des ponts et chaussées, 1833, p. 196,) the conditions of the equilibrium of a semi-arch ABNM, fig. 17, in respect to its tendency to revolve upon the edges of its voussoirs, either at the extrados or the intrados, when supported by a force Q acting perpendicular to the joint MN at its crown, are these: any joint *mn* being taken, the force Q must not be less than that which being applied at N would just prevent the portion M*mn*N, if it were a continuous solid, from turning upon the point *m*, and it must not be greater than that which being applied at M would be upon the point of causing it to turn about *n*; for if the first of these forces be called Q_1, and the second Q_2, then the force Q being less than Q_1 it is evident that if the arch did not turn upon some other point it would certainly turn upon the point *m*, and that this would be the case wherever in NM, Q was applied, since it would be insufficient to the equilibrium when applied in the most favourable position, which is N: again, if Q were greater than Q_2, then, provided the arch did not turn upon some other point of the extrados, it would certainly turn upon the point *n*, and that, wherever in MN it were applied, since when applied at the most unfavourable point M, it is sufficient to produce this revo-

lution. The force Q must then be less than one of the forces Q_1 and Q_2, and greater than the other. These conditions, however, only determine the stability of the semi-arch in respect to the particular joint *mn;* they assure us that rupture will not take place at that joint. Let corresponding values of Q_1 and Q_2 be found in respect to every joint; then if the value of Q be not less than any one of the series of values of Q_1 thus determined, and be not greater than any one of the values of Q_2, we are assured that the semi-arch will not turn upon the edges of any one of its joints.

Of all the values of Q_1 determined as above one will be the least; that least value equals the least force Q, which will support the semi-arch; it is therefore just equivalent to the tendency of the semi-arch to revolve towards the opposite semi-arch, and thus it is just equal to the pressure of the opposite semi-arch upon it.

This minimum of the forces Q_1 is thus, then (according to Coulomb), the actual horizontal thrust upon the key-stone of the arch; it must not exceed the maximum of the forces Q_2. This condition is necessary, and it is sufficient to the equilibrium of the arch, so far as the rotation of the voussoirs is concerned.

That particular joint in respect to which the least value of Q_1 is obtained is evidently the joint of rupture; it is there that if the horizontal pressure of the opposite semi-arch were rendered less than this value of Q, rupture would take place. Here then (there being no cement) the resultant of the forces upon MN*nm* passes actually through the point *m*, and the line of resistance

passes therefore through that point; so that the point of rupture, as determined by the theory of Coulomb, coincides with that determined by the theory given in the preceding pages. Moreover, the condition of the theory of Coulomb, that the force Q should not be less than one of the forces Q_1 and Q_2, or greater than the other, is evidently included in this other, that in respect to no joint *mn* of the structure the intersection with that joint of the resultant of the forces on MN*nm* should lie beyond the points *m* and *n*. Now this last condition is identical with that which is the fundamental condition of the equilibrium as determined by the preceding theory, viz., that the line of resistance should cut every joint actually within the mass of the structure. It is apparent that the minimum force Q_1 of the theory of Coulomb is identical with the minimum force P of the preceding theory.

So far as the determination of the position of the points of rupture and the amount of the horizontal thrust is concerned, the results of the two theories are the same; the difference is in the principles from which they start and the methods of investigation: that of Coulomb supposes a separate discussion of the conditions of the equilibrium of each particular voussoir, and as many distinct operations of analysis as there are voussoirs, and it determines the required maxima and minima by a comparison of the various different results thus obtained; the theory developed in the preceding pages includes the determination of all these in the discussion, by known methods of analysis, of a *single*

function, of the dimensions and form of the arch, and its loading. It here leaves the theory of Coulomb. A *single* analytical relation being established by it between the position of the points of rupture, the amount of the horizontal thrust, the dimensions of the arch, and the form and amount of the loading, these elements may be varied in any way with a reference to one another; the loading, for instance, may be varied in its distribution, so as to vary from one point to another the position of the points of rupture, or so as to vary, from one amount to another, the horizontal thrust.

From the determination of the horizontal thrust, and the position of the points of rupture, it proceeds to the complete determination of the line of resistance throughout its whole length; the condition that this line shall no where cut the extrados or back of the structure establishes the equilibrium in respect to any rotation of the voussoirs on their external edges, and determines the extreme amount of loading, placed on any point of the arch, which it may be made to bear. Traced to the abutment of the arch, the line of resistance ascertains the point where the direction of the resultant pressure intersects it, and the line of pressure fixes the actual direction of that resultant; these elements determine all the conditions of the equilibrium of the abutments, and therefore of the whole structure; they associate themselves directly with the conditions of the loading of the arch, and enable us so to distribute it as to throw the points of rupture into any given position on the

intrados, and give to the line of resistance any direction which shall best conduce to the stability of the structure; from known dimensions and a known loading of the arch they determine the dimensions of piers which will support it; or conversely from known dimensions of the piers they ascertain the dimensions and loading of the arch, which may safely be made to span the space between them.

CONDITIONS OF THE EQUILIBRIUM OF A STRUCTURE OF CEMENTED STONES.

The condition that the resultant pressure upon any portion of a structure must pass actually *through* the common surface of contact of that portion with the subjacent portion of the structure, on which condition the whole of the theory of the equilibrium of the structure has in the preceding pages been made to depend, is not a condition necessary to the equilibrium of a structure of *cemented* stones.

When the joints of the structure are cemented together, the resultant pressure may have its directions *beyond* the limits of any joint without causing the two portions separated by that joint to revolve upon the edges of one another, the distance of this deviation without the boundary of the joint being of course circumscribed within certain limits, imposed by the adhesive qualities of the cement, the dimensions of the adhering surface, and the inclinations and amounts of

insistent pressures. Thus then that essential condition of the discussion, according to which the line of resistance can no where *intersect* either the intrados or extrados of the arch, but which gives it a *contact* with the intrados at each of the points, called the points of rupture, altogether fails. A very simple consideration however brings this element, of the adhesion of the joints, into the discussion; and the theory is thus made to embrace the whole question of the stability of a cemented structure. It is this, that for the adhesion of the cement at each joint, supposed to operate uniformly over the whole surface of the joint, there may be substituted a force of the same amount acting at a single point on that surface, that point being its centre of gravity; because the adhesion thus uniformly distributed, and acting (so far as the tendency of the stones to turn over on their edges is concerned, which is the only case now under consideration) perpendicularly to the surface of the joint, may be considered to be made up of a system of equal and parallel forces, infinite in number and precisely analogous to the *weights* of the component elements of a thin uniform lamina, of the same form and dimensions as the joint; so that the system of elementary adhesions composing the adhesion of the joint shall have its resultant passing through exactly the *same point* as that through which the resultant of the weights of the elements of the lamina passes, that is, through the centre of gravity of the lamina.

In considering the conditions of the equilibrium of any

portion ASTD of a structure, fig. 14, and supposing the whole adhesive power of the joint ST to be called into operation, and to constitute an element of the equilibrium, we have only then to suppose that adhesion to be collected in the centre of gravity of ST and to act there in a direction perpendicular to the joint. This force being added to those which before entered into the discussion, viz., the weight of ASTD and the force P, will give a new direction and amount to the resultant R, which direction and amount are nevertheless determinable by the same methods of calculation. A new direction and amount of the force R for every position of S T brings with it a new form and direction of the line of resistance, and of the line of pressure. These are nevertheless determined, subject to the influence of these new elements of the calculation, precisely as before.

There is thus obtained the line of resistance and the line of pressure of the structure, on the supposition that the adhesive power of each of its joints is called into operation. Now on this hypothesis it is, as it was before, a necessary condition of the equilibrium, that the line of resistance shall pass actually through each joint of the structure, avoiding none; for if it avoid any joint, as for instance MN, fig. 2, then the resultant of all the forces acting upon the mass ACMN of the structure, there being included amongst these the whole amount of the adhesion of the surface MN, will act through some point *n* without that joint, and no force what-

ever opposing itself to the tendency of this resultant pressure to turn the mass over upon the point N, it will of necessity revolve upon that point.

The whole reasoning, this new element of adhesion being admitted into the discussion, becomes from this point, therefore, the same as before. Each semi-arch being supposed to require an opposite semi-arch to support it, so that neither would stand of itself, it follows that the line of resistance must somewhere in each semi-arch *meet* the intrados. Provided it does not also *cut* the extrados of the arch or of the pier, it will *touch* the intrados at the points where it thus meets it. The arch in this case will stand firmly. These points of contact of the line of resistance with the intrados are, as before, the points of rupture of the cemented arch: their position connects itself by a direct relation with the amount of the horizontal thrust, which amount is determined precisely as in the case of the uncemented arch. The case in which either semi-arch would stand without the support of an opposite semi-arch (which is the case of Mr. Brunel's experimental arch), is indicated by an evanescent or an impossible value of the horizontal thrust, resulting from the failure in this case of the hypothesis of a contact between the line of resistance and the intrados. It is evident that when the semi-arch will stand unsupported, no such contact takes place.

I have here shown by what process of calculation the adhesion of the cement of a structure may be included among the theoretical conditions of its equilibrium; but it has not been with the view of recommending the con-

sideration of this element in any practical question of construction. That structure (being of large dimensions) which would not stand without cement would assuredly be a perilous one.

A mass of masonry possessing few or none of those qualities which we understand under the term elasticity, the least appreciable alteration in the relative positions of its parts, such as not unfrequenly results from a settlement of its material, or an irregular sinking of its foundations, becomes a permanent disruption. No adhesive properties of the cement can control the disturbing forces called into operation whilst the arch is in the act of settling. In the case of an arch, such a disruption of the material is most likely to take place about the points of rupture. The adhesion of the cement is there then especially liable to be destroyed; and if this adhesion has been made an element in the calculation whence the proportions of the arch have resulted, so as to form a necessary condition to the equilibrium, then its destruction over these points of rupture will infallibly destroy the stability.[1]

That old principle of construction, then, which assigns to the structure such dimensions and proportions of its parts as would cause it to stand firmly were no cement used in its masonry, ought always to retain its authority with the judicious engineer.

[1] The destruction of the adhesive qualities of the cement over any point of the arch would *not* cause its overthrow, but only that over these points of rupture.

CONDITIONS OF THE EQUILIBRIUM OF A STRUCTURE OF CONCRETE, OF COURSES OF STONES, OR OF BRICK-WORK.

Hitherto this discussion of the conditions of the equilibrium of a structure of masonry has assumed it to be capable of disruption only in the given directions of the joints by which the single stones which compose it are in contact with one another, or where the cement is interposed between them.

This hypothesis does not however include the whole question of its stability. The stones or bricks which compose such a structure may themselves be capable of disruption, and being placed in layers or courses as the stones or bricks of an edifice usually are, they may be in contact not only by their superior and inferior surfaces but by their sides; all their faces being united by cement and their joints breaking one another. A structure thus formed is evidently capable of disruption, through any given point, in an *infinite number* of different directions, and, which of these is the direction of its actual rupture is an element of the theory to be determined.

From whatever point of the surface of the structure the rupture may commence, its direction is subject to this condition, that it is that direction in which the resistance to rupture—as opposed to the particular forces which in each case tend to produce it—is the least; or, in other words, supposing the rupture to take place through any given point in the intrados or extrados

of the structure, then will the surface of rupture be that surface passing across the structure through this point, over which the various resistances opposing themselves to the rupture are the *least*. Thus, for instance, in fig. 18, if N be a point in the intrados of an arch about which rupture is about to take place, then will the direction of this rupture be such that the surface of rupture MN shall be that surface passing through N, on which the resistances opposing themselves to the rupture are, in their aggregate effect, the least. In a structure homogeneous as to the qualities of its materials and their distribution, this surface of least resistance to rupture is nearly a *plane*;[1] for let M and N be points on the opposite side of the surface of rupture, then since the shortest line which can be drawn between N and M is the *straight* line between them, supposing the resistance to rupture to act uniformly over the surface of rupture, that resistance will be least between M and N, when the surface of rupture coincides with the straight line MN; and the same being true for all other points similarly situated with M and N, it is evident that the surface of least resistance to rupture is a surface which may be generated by the motion of a straight line.

This would be *strictly* true were there any case of

[1] This remark does not extend to the case of the rupture by transverse strain of masses of greater compressibility than masonry, as for instance, masses of wood and iron in which the neutral axis lies, at the instant of rupture, *within* the surface of the mass, one side of it yielding by the compression, whilst the other yields by the extension of its material.

a structure throughout which the various forces opposing themselves to rupture were distributed with perfect uniformity. A structure of concrete would approach nearest to this case. In a cemented structure of the ordinary construction the surfaces of least resistance follow, for the most part, the joints of the bricks or stones, the adhesion of these to one another by their joints being less, where the interposed cement is mortar, than the adhesion of the individual parts of each brick or stone to one another. Subject to this condition the surface of least resistance still however approaches as nearly as possible to a plane surface when the materials of the structure are uniformly distributed.

Where the materials are not uniformly distributed, as in the arch, fig. 19, where the joints of the voussoirs have one direction, and those of the mass of masonry which it supports another, it is evident that the surface of least resistance to rupture, which follows the joint of the voussoir from N to M, will above the point M take a new direction ML, forming on the whole a line of broken inclination NML.

To simplify the discussion of these new elements of the theory, let there be supposed a uniform arrangement of the materials of the structure, and a uniform distribution of the forces resisting its rupture, as in the case of a structure of concrete, any deviations from this law which may occur in practice will be in favour of its stability, if we assume the unit of resistance to rupture at the least value which any part of the structure supplies.

Let any plane MN, fig. 18, be imagined to intersect the mass, inclined at an angle θ to the horizon. Let the line of resistance be determined as in the case of an uncemented structure; if this line pass actually through the section MN there will be no tendency to rupture by the turning of either portion of the mass about the edge of that section. Suppose however that it does not, and that its direction is so far beyond these limits that the whole resistance to rupture on the plane MN is called into operation. This resistance may be estimated in terms of the unit of resistance, the inclination θ of the plane of intersection, and the ordinate BO of the point N. The mass ABNM being imagined to be acted upon by this resistance, its weight, and the pressure P, let the resultant of these three forces, and its point of intersection with the plane MN be determined, and the plane being then conceived to take up an infinity of different positions by the variation of BO, let the *locus* of these intersections be found; let then P be assumed to have such a value as to give to this line of resistance a common point with the surface of the mass. This condition will serve to determine a relation between the force P, the ordinate BO of the point N, where the line of resistance meets the surface, and the inclination θ of the intersecting plane. If the rupture of the structure be not about to take place at some point above N, it will evidently be upon the point of taking place at N, along a plane whose inclination is θ. But to every value of θ corresponds a different value of the ordinate BO, and a different position of N subject to these con-

ditions ; so that at some point above N the rupture may be about to take place along a plane inclined at some other angle θ to the horizon. Determine then the *minimum* value of the ordinate BO in respect to the variable θ of which it is a function, or the least height at which any inclination of the plane of rupture can cause an intersection between the line of resistance and the surface of the structure, the corresponding point N will be the true point of rupture ; above that point the parts of the structure will not be separated by the pressures to which they are subjected, but at that point they will be upon the point of separating. This point determined, the discussion of the further conditions of the equilibrium is precisely the same as before. If the line of resistance thus determined cut the surface of the structure at N, actual rupture will take place ; if it merely touch, as in the case of the arch, there will be no rupture, but a state preserved which is always one bordering on rupture. The determination of this point has however supposed a knowledge of the circumstances under which rupture may take place along the plane MN of rupture, and of the unit of force opposing itself to rupture on that plane. The circumstances of this rupture are of two kinds. It may take place along the plane MN by the *turning* of the mass ABNM upon the point N, the forces opposing themselves to which rupture are the direct adhesions of the parts of the mass along that plane and perpendicular to it, and this is the case above supposed ; or it may take place by the sliding of the mass ABNM′ along the plane of rupture NM′, to which

rupture are opposed, 1st, those *cohesive* forces of the mass which oppose themselves to the sliding of any two portions of it asunder, and which are independent of the insistent pressures; and, 2ndly, the forces of friction which oppose themselves to the motion of any two portions of the mass upon one another when separated, and which are dependent upon the insistent pressures.

Now the forces thus entering into the discussion of both these cases admit of determination in terms of known units of each, and may therefore be made to enter with precision into the formulæ of equilibrium.

In the consideration of the forces which oppose themselves to rupture in the first case, where they act in directions perpendicular to the plane MN, there is a difficulty in making the proper estimate of those *compressible* and *elastic* properties which are common in different degrees to all bodies, but which may, perhaps, be considered to be called less actively into operation in a mass of masonry than in any other. We may at any rate assign limits within which their influence on the stability of a structure must be confined. Let us first suppose the adhesion of the surfaces of contact in MN not to be at all affected by the elastic properties of the material; the resultant of all the elementary adhesions of the parts of that plane will then pass through its centre of gravity, or, if it be a rectangle, as is commonly the case, the resultant will pass through the centre of that rectangle. Next let us suppose the surfaces of contact in MN to be united by a stratum of some perfectly elastic material, a lamina of caoutchouc

for instance; the extension of this elastic material will then be different at different distances from N, varying, when the surfaces turn upon N, directly at these distances; and in the same proportion will vary the forces by which at different points the two surfaces are held together. Now from this it may readily be calculated by the principles of the integral calculus, that the point through which the resultant of all these elementary elastic forces, holding the two surfaces together, passes, is no longer situated in the centre of the rectangle, but at a distance of $\frac{1}{6}$th the distance MN beyond that centre. So that supposing the aggregate adhesion to be in this case the same as before, its efficiency to maintain the equilibrium will be increased by an increased leverage of this amount. Now between these two limits of perfect elasticity and compressibility, and perfect rigidity of the material opposing its adherence to rupture in MN, the true case of the adherence of a mass of masonry may be considered to lie. The stability will certainly not be less than that given by the first hypothesis, and not greater than that given by the second. It is because the first hypothesis thus assigns the lowest estimate to the influence on the stability of adhesive properties in the cement, that I have assumed it as the basis of the method explained in my last paper (page 29), for determining the conditions of the equilibrium of a cemented structure of single stones.

To determine the conditions of rupture in the second case where the structure yields by sliding upon the plane NM′, assume any inclination θ to the plane of rupture

NM′, fig. 18, determine the direction of the resultant of the pressures upon ABNM′, including amongst these pressures the pressure P, the weight of the mass AB NM′, and the forces of adhesion which oppose themselves to the sliding of the surfaces of contact in NM′ upon one another independently of the friction. If the inclination of this resultant to the perpendicular to NM′ be greater than the limiting angle of resistance, rupture will take place. Suppose it to equal this angle so that rupture may be upon the point of taking place. A relation will thus be determined between θ and BO the ordinate of the point N, so that for that position of N rupture would be upon the point of taking place along a plane whose inclination is θ. The minimum value of BO, considered as a function of θ which satisfies this condition, determines the point of rupture. At that point two portions of the mass will slide upon one another, and at no point above it.

The two cases will give different geometrical forms to the lines of resistance and different positions of the point of rupture N. The true point of rupture of the two thus determined, will be the highest.

In the great majority of structures, however, we may be assured that the first case is the only one which it is necessary to consider, rupture by the sliding asunder of two portions of the mass across its solid material rarely occurring.

THE MATHEMATICAL THEORY OF THE EQUILIBRIUM OF A STRUCTURE.

In order that the principles laid down in the preceding papers may become intelligible when translated into the language of analysis under their more general forms, it will be well, first of all, to present to the reader some of their simpler and more elementary applications.

I propose then, *first*, to consider the equilibrium of the PIER; *secondly*, that of the loaded CIRCULAR ARCH with equal voussoirs; *thirdly*, I propose to discuss those *general methods of analysis* by which the lines of resistance and pressure may be determined in ANY STRUCTURE subject to *any* given circumstances of pressure, and whence may be deduced the actual conditions of the equilibrium of that structure. Of this last determination, all the others are, of course, particular cases, and the order of investigation which I here propose to myself is the opposite of that which I followed in the original consideration of the question.

THE PIER.

Let ABCD, fig. 20, represent an upright pier of uniform dimensions to any point in the summit AB, of which is applied a given pressure P. To determine the line of resistance in this pier let MN be conceived to be any horizontal section of it. The forces impressed upon the portion AMNB of the pier are then, its weight

W, which may be conceived to be collected in the direction of its axis and applied at S, and the pressure P on the summit of the pier. Let R be the resultant of W and P, then is L, where the direction of R intersects MN, a point in the line of resistance. Let $ES=x$, $SL=y$, a relation established between x and y will determine the position of the point L for every position of the intersecting plane, and will be the equation to the line of resistance.

Let AB, the width of the pier, be represented by 2K, and the weight of a cubical unit of the pier by μ. Let the distance EQ of the point of application of the force P from the axis of the pier be k, and the inclination of PQ to the vertical Φ; also let the inclination of R to the vertical be θ.

Then the forces P, W, R, being resolved vertically and horizontally become

$$\left.\begin{matrix} P\cos.\Phi, & W, & R\cos.\theta \\ P\sin.\Phi, & & R\sin.\Phi \end{matrix}\right\}$$

therefore, by the general conditions of the equilibrium of forces in the same plane, observing that the coordinates of the points of application of P, W, and R, are respectively

$$\begin{matrix} -k, 0 \\ 0, +x \\ +y, +x \end{matrix}$$

$$\left.\begin{matrix} P\cos.\Phi+W-R\cos.\theta=0 \\ P\sin.\Phi-R\sin.\theta=0 \\ -k\,P\cos.\Phi-Ry\cos.\theta+Rx\sin.\theta=0 \end{matrix}\right\} \ldots\ldots (1)$$

Whence, observing that $W=2K\mu x$, we obtain by the elimination of R and θ in the last equation

$$-kP\cos.\Phi-y\,(P\cos.\Phi+2K\mu x)+Px\sin.\Phi=o$$

$$\therefore\ y=P.\frac{x\sin.\Phi-k\cos.\Phi}{2K\mu x+P\cos.\Phi}\ \ldots\ldots\ldots\ (2)$$

which is the required equation to the line of resistance.

SECOND METHOD OF DETERMINING THE LINE OF RESISTANCE.

To persons unaccustomed to the application of the general equations of equilibrium the following method of determining the line of resistance in a pier will be more intelligible.

Take OU=SE (fig. 21) to represent the weight W of the mass ABNM. Produce PQ to V, and take OV to represent the force P on the same scale on which OU represents W. Complete the parallelogram OUYV, then OY will represent the resultant R in magnitude and direction, and L will be a point in the line of resistance.

Draw XY parallel to SL. Therefore by similar triangles,

$$\frac{SL}{OS}=\frac{XY}{OX}=\frac{UY\sin.XUY}{OU+UY\cos.XUY}$$

$$=\frac{P\sin.XUY}{W+P\cos.XUY}\ \ldots\ldots\ldots\ (1)$$

Also adopting the same notation as before

$SL=y, OS=ES-EO=x-k\cot.\Phi, XUY=\Phi, W=2K\mu x$

$$\therefore \frac{y}{x-k\cot.\Phi}=\frac{\mathrm{P}\sin.\Phi}{2\mathrm{K}\mu x+\mathrm{P}\cos.\Phi}$$

$$\therefore y=\mathrm{P}\frac{x\sin.\Phi-k\cos.\Phi}{2\mathrm{K}\mu x+\mathrm{P}\cos.\Phi} \quad \ldots\ldots\ldots (2$$

If P′ represent the solidity of a mass of the same material as the pier whose weight shall equal P, then

$$\mathrm{P}=\mathrm{P}'\mu$$

$$\therefore y=\mathrm{P}'.\frac{x\sin.\Phi-k\cos.\Phi}{2\mathrm{K}x+\mathrm{P}'\cos.\Phi} \quad \ldots\ldots\ldots (3$$

Now this equation is that to a rectangular hyperbola, as will be apparent from the following transformations.

$$y\left\{2\mathrm{K}x+\mathrm{P}'\cos.\Phi\right\}=\mathrm{P}'x\sin.\Phi-\mathrm{P}'k\cos.\Phi$$

$$\therefore y\left\{x+\frac{\mathrm{P}'\cos.\Phi}{2\mathrm{K}}\right\}-\frac{\mathrm{P}'\sin.\Phi}{2\mathrm{K}}x=-\frac{\mathrm{P}'k\cos.\Phi}{2\mathrm{K}}$$

Subtracting $\frac{\mathrm{P}'^2\sin.\Phi\cos.\Phi}{4\mathrm{K}^2}$ from both sides

$$y\left\{x+\frac{\mathrm{P}'\cos.\Phi}{2\mathrm{K}}\right\}-\frac{\mathrm{P}'\sin.\Phi}{2\mathrm{K}}\left\{x+\frac{\mathrm{P}'\cos.\Phi}{2\mathrm{K}}\right\}$$
$$=-\frac{\mathrm{P}'\cos.\Phi}{2\mathrm{K}}\left\{\frac{\mathrm{P}'\sin.\Phi}{2\mathrm{K}}+k\right\}$$

$$\therefore\left\{\frac{\mathrm{P}'\sin.\Phi}{2\mathrm{K}}-y\right\}\left\{x+\frac{\mathrm{P}'\cos.\Phi}{2\mathrm{K}}\right\}=\frac{\mathrm{P}'\cos.\Phi}{2\mathrm{K}}$$
$$\left\{\frac{\mathrm{P}'\sin.\Phi}{2\mathrm{K}}+k\right\}$$

Let EG, fig. 22, be taken $=\frac{\mathrm{P}'\sin.\Phi}{2\mathrm{K}}$

GH . . . . . . $=\frac{\mathrm{P}'\cos.\Phi}{2\mathrm{K}}$

ES$=x$ SL$=y$

$$\therefore \text{IL} = \frac{\text{P}' \sin.\Phi}{2\text{K}} - y$$

$$\text{IH} = \frac{\text{P}' \cos.\Phi}{2\text{K}} + x$$

$$\therefore \text{IL} \cdot \text{IH} = \left\{\frac{\text{P}' \sin.\Phi}{2\text{K}} - y\right\} \left\{\frac{\text{P}' \cos.\Phi}{2\text{K}} + x\right\}.$$

$$= \frac{\text{P}' \cos.\Phi}{2\text{K}} \left\{\frac{\text{P}' \sin.\Phi}{2\text{K}} + k\right\}$$

= a constant for all the joints of the same pier subjected to the same insistent pressures.

Since then the rectangle of the co-ordinates of the line of resistance in respect to the axis HX is for every point of that line the same, it follows, by a well known property, that its geometrical form is that of a rectangular hyperbola, of which HX is the asymptote.

The line of resistance therefore continually *approaches* HX, but never meets it. If then HX lies, as shown in the figure, actually *within* the mass of the pier, or if EG be less than EB or $\frac{\text{P}' \sin.\ \Phi}{2\ \text{K}}$ less than K, or P′ sin. Φ than 2 K^2, then the line of resistance will no where cut the extrados of the pier, and the structure will retain its stability under the insistent pressure P, however high it may be built.

If HK lie *without* the mass of the pier, or P′ sin. Φ be not less than 2 K^2, then the line of resistance will somewhere cut the intrados of the pier, provided it be built of a sufficient height, and this height will evidently be the greatest to which the pier can be built without being overthrown ; it is evidently equal to that value of x

which is determined in the equation 3 to the line of resistance by assuming y equal to K, or to one-half the width of the pier.

This substitution being made, and the corresponding value of x, or the greatest possible height of the pier, being represented by H, we have

$$2\,K^2\,H + P'\,K\cos.\,\Phi = P'\,H\sin.\,\Phi - P'\,k\cos.\,\Phi$$

$$\therefore H = \frac{(K+k)\,P'\cos.\,\Phi}{P'\sin.\,\Phi - 2\,K^2} \quad \ldots\ldots\ldots \quad (4)$$

It will be observed that P sin. Φ and P cos. Φ are the resolved parts, horizontally and vertically, of the insistent pressure P, and that P′ sin. Φ and P′ cos. Φ are the volumes of the same material as the pier, whose weights would be equal to these resolved pressures; moreover, that if these volumes be supposed to be those of masses of the same section with the pier, then their *heights* will be represented by $\frac{P'\sin.\,\Phi}{2\,K}$ and $\frac{P'\cos.\,\Phi}{2\,K}$.

RULE TO DETERMINE, BY CONSTRUCTION, THE DIRECTION AND AMOUNT OF THE RESULTANT PRESSURE UPON ANY JOINT OF A PIER.

Let MN, fig. 21, be any joint of the pier. And PQ the resultant of the pressures which it bears. Produce PQ to V intersecting the axis EX in O. Take OU =ES, and OV equal to the height of a mass, which being of the same section and material as the pier, shall have a weight equal to the pressure P. Complete the parallelogram OUYV, and draw its diagonal OY, then

will this line be in the direction of the resultant of the pressures upon the joint MN; its length will, moreover, equal the height of a mass of the same section and material as the pier, the weight of which mass would just equal the amount of this resultant pressure.

It is evident, since the resultant pressures on all the joints pass through the point O, and have their directions between the lines OV and OX, that they are all inclined to the vertical at less angles than the force P; so that if the direction of PQ be not without the limiting angle of resistance, then will the directions of the resultants of the pressures on none of the joints be without that angle, so that none of the stones of the pier will be made to slip upon one another; it can only therefore fall by being made to turn on the edges of some of its subjacent stones. The truth of the above rule is evident from the theory given in page 43.

RULE TO DETERMINE, BY CONSTRUCTION, WHETHER A PIER BUILT UP OF SINGLE STONES, WITHOUT CEMENT, WILL STAND UNDER THE PRESSURES TO WHICH IT IS SUBJECTED, TO WHATEVER HEIGHT IT MAY BE BUILT.[1]

Resolve the pressures sustained by the pier[2] vertically

[1] This and the following rules are extracted from the author's work entitled "Illustrations of Mechanics," page 209.

[2] It will be observed that if in fig. 21 the distance EQ or k exceed EA, the pressure P will be applied on the face or intrados AM of the pier; this case is therefore fully included in the results of the preceding discussion.

and horizontally. Calculate the height of a mass, which being of the same substance and the same section as the pier, shall have a weight equal to the sum of those pressures which are thus resolved horizontally. Let AS, fig. 23, be this height. From the centre E of the pier measure EG equal to AS, and draw the vertical line CGX; this line will be the asymptote to the line of resistance, which has been shown to be a hyperbola. If GX lie within the mass of the structure, or if EG be less than EB, or less than half the width of the pier, then the structure will stand, to however great a height it may be built. If EG be greater than EB, so that GX lies *without* the structure, then the pier cannot be built above a certain height, so as to stand under the insistent pressures.

RULE TO DETERMINE BY CONSTRUCTION THE GREATEST HEIGHT TO WHICH A PIER CAN BE BUILT SO AS TO SUSTAIN GIVEN INSISTENT PRESSURES.

Let P, fig. 24, represent the point where the resultant of the insistent pressures intersects the summit of the pier, and let AS and AT represent the heights of masses which, being of the same section and the same material as the pier, shall equal in weight respectively, the sum of the insistent pressures when resolved horizontally, and their sum when resolved vertically.

Take EG equal to AS, E being the centre of the pier, and let the point G be beyond B, so that the pier will (by the last rule) be overthrown, if raised above a certain

height to be determined; join G U, and through the point P draw PZ parallel to GU, then will BZ be the greatest height to which the pier can be raised, or if it be carried higher, then is this the point to which an inclined buttress should be built to support it.[1]

TO DETERMINE THE LINE OF RESISTANCE IN AN ARCH WHOSE INTRADOS IS A CIRCLE, WHOSE EXTRADOS IS OF ANY GIVEN GEOMETRICAL FORM, AND WHICH SUPPORTS ANY GIVEN LOAD.

Let ADBF, fig. 25, represent any portion of such an arch, P a pressure applied at its extreme voussoir, and X and Y the horizontal and vertical components of any pressure borne upon its extrados, or of the *resultant* of any number of such pressures; let moreover the co-ordinates, from the centre C, of the point of application of this pressure, or this *resultant* pressure, be x and y.

Let the horizontal force P be applied in AD at a vertical distance p from C; also let CT represent any plane which, passing through C, intersects the arch in a direction parallel to the joints of its voussoirs.

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[1] For by similar triangles

$$\frac{BZ}{BP}=\frac{BU}{BK}=\frac{AT}{AS-EB},$$

$$\therefore \frac{BZ}{(K+k)}=\frac{\left(\frac{P' \cos. \Phi}{2K}\right)}{\left(\frac{P' \sin. \Phi}{2K}-K\right)};$$

$$\therefore BZ=\frac{(K+k)\,P' \cos. \Phi}{P' \sin. \Phi - 2K^2}=H' \text{ by equation (4).}$$

Let this plane be intersected by the *resultant* of the pressures applied to the mass ASTD in R. These pressures are the weight of the mass ASTD, the load X and Y, and the pressure P. Now if pressures equal and parallel to these, but in opposite directions, were applied at R, they would of themselves support the mass, and the whole of the subjacent mass TSB might be removed without affecting the equilibrium. Imagine this to be done; call M the weight of the mass ASTD, and h the horizontal distance of its centre of gravity from C, and let CR be represented by ρ, and the angle ECS by θ, then the perpendicular distances from C of the pressures $M+Y$ and $P-X$, imagined to be applied to R, are $\rho \sin. \theta$ and $\rho \cos. \theta$; therefore by the condition of the equality of moments

$$(M+Y)\rho \sin.\theta+(P-X)\rho \cos.\theta=Mh+Yx-Xy+Pp. \quad (5)$$

$$\therefore \rho=\frac{Mh+Yx-Xy+Pp}{(M+Y)\sin.\theta+(P-X)\cos.\theta},$$

which is the equation to the line of resistance.

M and h are given functions of θ, as also are X and Y, if the pressure of the load extend *continuously* over the surface of the extrados from D to T.

It remains from this equation to determine the pressure P, being that supplied by the opposite semi-arch. As the simplest case, let all the voussoirs of the arch be of the same depth, and let the inclination ECP of the first joint of the semi-arch to the vertical be represented by Θ, and the radii of the intrados and extrados by R and r. Then by the known principles of statics,

$$\mathrm{M}\,h=\int_r^{\mathrm{R}}\int_{\Theta}^{\theta} r^2 \sin.\,\theta\, d\theta\, dr=-\tfrac{1}{3}\,(\mathrm{R}^3-r^3)\,(\cos.\,\theta-\cos.\,\Theta),\ \text{also } \mathrm{M}=\tfrac{1}{2}\,(\mathrm{R}^2-r^2)\,(\theta-\Theta)$$

$$\therefore \rho\,\left\{\tfrac{1}{2}\,(\mathrm{R}^2-r^2)\,(\theta-\Theta)\sin.\,\theta+\mathrm{Y}\sin.\,\theta-\mathrm{X}\cos.\,\theta+\mathrm{P}\cos.\,\theta\right\}=\tfrac{1}{3}\,(\mathrm{R}^3-r^3)\,(\cos.\,\Theta-\cos.\,\theta)+\mathrm{Y}\,x-\mathrm{X}\,y+\mathrm{P}\,p \quad (6)$$

Now at the points of rupture the line of resistance *meets* the intrados, so that there $\rho=r$; if then Ψ be the corresponding value of θ,

$$r\left\{\tfrac{1}{2}\,(\mathrm{R}^2-r^2)\,(\Psi-\Theta)\sin.\,\Psi+\mathrm{Y}\sin.\,\Psi-\mathrm{X}\cos.\,\Psi+\mathrm{P}\cos.\,\Psi\right\}=\tfrac{1}{3}\,(\mathrm{R}^3-r^3)\,(\cos.\,\Theta-\cos.\,\Psi)+\mathrm{Y}\,x-\mathrm{X}\,y+\mathrm{P}\,p\ .\ (7)$$

Also at the points of rupture the line of resistance *touches* the intrados, so that there $\frac{d\rho}{d\theta}=\frac{dr}{d\theta}=0$; assuming then, to simplify the results, that the presure of the load is wholly in a vertical direction, so that $\mathrm{X}=0$, and that it is collected over a single point of the extrados, so that $\frac{d\mathrm{Y}}{d\theta}=0$, and differentiating equation (6), and assuming $\frac{d\rho}{d\theta}=0$ when $\theta=\Psi$ and $\rho=r$, we obtain

$$r\left\{\tfrac{1}{2}\,(\mathrm{R}^2-r^2)\,(\Psi-\Theta)\cos.\,\Psi+\tfrac{1}{2}\,(\mathrm{R}^2-r^2)\sin.\,\Psi+\mathrm{Y}\cos.\,\Psi-\bar{\mathrm{P}}\sin.\,\Psi\right\}=\tfrac{1}{3}\,(\mathrm{R}^3-r^3)\sin.\,\Psi\,;$$

hence, assuming $\mathrm{R}=r\,(1+\alpha)$,

$$\left\{\frac{6\,\mathrm{P}}{r^2}+\alpha^2\,(2\,\alpha+3)\right\}\tan.\,\Psi=\left\{\frac{6\,\mathrm{Y}}{r^2}-3\alpha\,(\alpha+2)\,\Theta\right\}+3\,\alpha\,(\alpha+2)\,\Psi\ \ldots\ldots\ldots\ (8)$$

Eliminating $(\Psi-\Theta)$ between equations (7) and (8), we have

$$\left\{\frac{\mathrm{P}}{r^2}+\alpha^2\,(\tfrac{1}{3}\,\alpha+\tfrac{1}{2})\right\}\sec.\,{}^2\Psi-\left\{\frac{\mathrm{Y}\,x+\mathrm{P}\,p}{r^3}+\alpha\,(\tfrac{1}{3}\,\alpha^2+\alpha+1)\cos.\,\Theta\right\}\sec.\,\Psi=-\alpha\,(\tfrac{1}{2}\alpha+1)\ \ldots\ldots\ (9)$$

Eliminating P between equations (7) and (8), and reducing

$$\frac{Y^*}{r^2}\left\{\frac{p\cos.\Psi+x\sin.\Psi}{r}-1\right\}=(\tfrac{1}{2}\alpha^2+\alpha)\left(1-\frac{p}{r}\cos.\Psi\right)(\Psi-\Theta)+\frac{p}{r}(\tfrac{1}{2}\alpha^2+\tfrac{1}{3}\alpha^3)\sin.\Psi-$$
$$\left\{(\alpha+\alpha^2+\tfrac{1}{3}\alpha^3)\cos.\Theta-(\tfrac{1}{2}\alpha^2+\alpha)\cos.\Psi\right\}\sin.\Psi \quad\ldots\ldots\ldots \quad (10)$$

Let A P, fig. 25, $=\lambda r$ $\therefore$ $\frac{p}{r}=(1+\lambda)\cos.\Theta$. Substituting this value of $\frac{p}{r}$,

$$\frac{Y}{r^2}\left\{\frac{x}{r}\sin.\Psi+(1+\lambda)\cos.\Theta\cos.\Psi-1\right\}=(\tfrac{1}{2}\alpha^2+\alpha)\left\{\{1-(1+\lambda)\cos.\Theta\cos.\Psi\}(\Psi-\Theta)+(\cos.\Psi-\cos.\Theta)\sin.\Psi\right\}$$
$$+\lambda(\tfrac{1}{2}\alpha p+\tfrac{1}{3}\alpha^3)\sin.\Psi\cos.\Theta \quad\ldots\ldots\ldots \quad (11)$$

If the arch be a continuous segment the joint AD is vertically above the centre, and CD coinciding with CE, $\Theta=o$; if it be

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* This equation might have been obtained by differentiating equation (6) in respect to P and θ, and assuming $\frac{dP}{d\theta}=0$ when r and Ψ are substituted for ρ and θ; for if that equation be represented by $u=0$, u being a function of P, ρ and θ, $\frac{du}{dP}\frac{dP}{d\theta}+\frac{du}{d\theta}=0$, and $\frac{du}{d\rho}\frac{d\rho}{d\theta}+\frac{du}{d\theta}=0$. The same result $\frac{du}{d\theta}=0$ is therefore obtained whether we assume $\frac{dP}{d\theta}=0$, or $\frac{d\rho}{d\theta}=0$, which last supposition is that made in equation (8), whence equation (10) has resulted. The hypotheses $\frac{dP}{d\theta}=0$, $\rho=r$, determine the minimum of the pressures P, which being applied to the key-stone will prevent the semi-arch from turning on any of the successive joints of its voussoirs.

a broken segment, as in the Gothic arch, Θ has a given value determined by the character of the arch. In the pure or equilateral Gothic arch $\Theta=30^\circ$. Assuming $\Theta=0$, and reducing

$$\frac{\mathrm{Y}}{r^2}\left\{\frac{x}{r}-\left(\tan.\frac{\Psi}{2}-\lambda\cot.\Psi\right)\right\}=\left(\tfrac{1}{2}\alpha^2+\alpha\right)\left\{\left(\tan.\frac{\Psi}{2}-\lambda\cot.\Psi\right)\Psi-\text{vers.}\,\Psi\right\}+\lambda\left(\tfrac{1}{2}\alpha^2+\tfrac{1}{3}\alpha^3\right)\quad\ldots\ldots\;(12)$$

It may easily be shown that as Ψ increases in this equation Y increases and conversely, so that as the load is increased the points of rupture descend. When $\mathrm{Y}=0$, or there is no load upon the extrados,

$$\left(\tan.\frac{\Psi}{2}-\lambda\cot.\Psi\right)\Psi-\text{vers.}\,\Psi+\tfrac{1}{3}\lambda\alpha\,\frac{3+2\alpha}{2+\alpha}=0\quad\ldots\ldots\;(13)$$

When $x=0$, or the load is placed on the crown of the arch,

$$\frac{\mathrm{Y}}{r^2}=\frac{\left(\tfrac{1}{2}\alpha^2+\alpha\right)\text{vers.}\,\Psi-\lambda\left(\tfrac{1}{2}\alpha^2+\tfrac{1}{3}\alpha^3\right)}{\tan.\frac{\Psi}{2}-\lambda\cot.\Psi}-\left(\tfrac{1}{2}\alpha^2+\alpha\right)\Psi\quad\ldots\ldots\;(14)$$

When $\frac{x}{r}-\left(\tan.\frac{\Psi}{2}-\lambda\cot.\Psi\right)=0,\ \frac{\mathrm{Y}}{r^2}$ becomes infinite; an infinite load is therefore required to give that value to the angle of rupture which is determined by this equation. Solved in respect to $\tan.\frac{\Psi}{2}$, it gives

$$\tan.\frac{\Psi}{2}=\frac{\left(\frac{x}{r}\right)+\sqrt{\left(\frac{x}{r}\right)^2+\lambda\,(2+\lambda)}}{2+\lambda}\quad\ldots\ldots\;(15)$$

No loading placed upon the arch can cause the angle of rupture to exceed that determined by this equation.

Let us now consider the case, in which the pressure of the load is distributed over different points of the extrados ; let it be supposed that this pressure is wholly vertical, and such that any portion F T of the extrados (fig. 29) sustains the weight of a mass G F T V immediately superincumbent to it, and bounded by the straight line G V inclined to the horizon at the angle ι; let moreover the weight of each cubical unit of the load be equal to that of the same unit of the material of the arch, multiplied by the constant factor μ; then representing A D by R β, we have

$$\mathrm{Y}=\text{mass GFTV}=\mu \mathrm{R}^2\int_\Theta^\theta \left\{1+\beta-\sec.\iota\cos.(\theta-\iota)\right\}\cos.\theta\, d\theta=\mu\mathrm{R}^2\left\{(1+\beta)(\sin.\theta-\sin.\Theta)-\tfrac{1}{4}\sec.\iota\{\sin.(2\theta-\iota)\right.$$
$$\left.-\sin.(2\Theta-\iota)\}-\tfrac{1}{2}(\theta-\Theta)\right\} \quad \ldots\ldots \quad (16)$$

$$\mathrm{Y}x=\text{momentum of GFTV}=\mu\mathrm{R}^3\int_\Theta^\theta\left\{(1+\beta)-\sec.\iota\cos.(\theta-\iota)\right\}\sin.\theta\cos.\theta\, d\theta=\mu\mathrm{R}^3\left\{\tfrac{1}{2}(1+\beta)(\cos.^2\Theta-\cos.^2\theta)\right.$$
$$\left.-\tfrac{1}{3}(\cos.^3\Theta-\cos.^3\theta)-\tfrac{1}{3}\tan.\iota(\sin.^3\theta-\sin.^3\Theta)\right\} \quad \ldots\ldots \quad (17)$$

As the simplest case let us suppose D V horizontal, and the crown of the arch to be at A, so that $\iota=0$, and $\Theta=0$. Substituting the values of Y and Y x which result from these suppositions, in equation (7), solving that equation in respect to $\frac{\mathrm{P}}{r^2}$ and reducing, we have

$$\frac{\mathrm{P}}{r^2}=\frac{(1-\alpha)(1+\alpha)^2(1+\beta)\sin.^2\Psi+\frac{1}{6}(1+\alpha)^2(1-2\alpha)\cos.^3\Psi+(\frac{1}{2}\alpha^2+\frac{1}{3}\alpha^3-\frac{1}{2})\cos.\Psi-\frac{1}{2}\Psi\sin.\Psi+\frac{1}{3}}{1+\lambda-\cos.\Psi} \quad \ldots \quad (18)$$

Assuming $\frac{d\mathrm{P}}{d\Psi}=0$ (see note, page 52), and $\lambda=\alpha$, and reducing

$$\tfrac{2}{3}(1-2\alpha)(1+\alpha)^2\cos.^3\Psi-\left\{(1-\alpha)(1+\beta)+(1+\alpha)(1-2\alpha)\right\}\cos.^2\Psi+\left\{\frac{1}{(1+\alpha)^2}+2(1-\alpha^2)(1+\beta)\right\}\cos.\Psi$$

$$+\frac{1}{(1+\alpha)^2}\left\{1-(1+\alpha)\cos.\Psi\right\}\frac{\Psi}{\sin.\Psi}-(1-\alpha)\beta-\frac{2}{3(1+\alpha)^2}-\frac{1+\frac{2}{3}\alpha^3}{1+\alpha}=0 \quad \ldots\ldots\ldots \quad (19)$$

In the case in which the line of resistance passes through the bottom of the key-stone so that $\lambda=0$, equation 18 becomes

$$\frac{P}{r^2}=\tfrac{1}{3}(1+\alpha)^3(1+\beta)(1-\alpha)(1+\cos.\Psi)-\tfrac{1}{6}(1+\alpha)^3(1-2\alpha)(1+\cos.\Psi)\cos.\Psi-\tfrac{1}{2}\Psi\cot.\tfrac{1}{2}\Psi+\tfrac{1}{3}=0 \quad \ldots \quad (19)$$

whence, assuming $\frac{dP}{d\Psi}=0$, we have

$$\tfrac{2}{3}(1+\alpha)^2(1-2\alpha)\cos.^2\Psi+(1+\alpha)^2\left\{\tfrac{1}{3}\alpha-(1-\alpha)\beta-\tfrac{1}{3}\right\}\cos.\Psi+\frac{\Psi}{\sin.\Psi}+1=0 \quad \ldots\ldots\ldots \quad (20)$$

Let us next take a case of *oblique* pressure on the extrados, and let us suppose it to be the pressure of *water*, whose surface stands at a height β R above the summit of the key-stone. The pressure of this water being perpendicular to the extrados will every where have its direction through the centre C, so that its moment about that point will vanish, and $Yx-Xy=0$; moreover, by the principles of hydrostatics, the vertical component Y of the pressure of the water, superincumbent to the portion A T of the extrados, will equal the weight of that mass of water, and will be represented by the formula (16), if we assume $\iota=0$. The horizontal component X of the pressure of this mass of water is represented by the formula

$$X=\mu R^2\int_{\Theta}^{\theta}\left\{1+\beta-\cos.\theta\right\}\sin.\theta\,d\theta=\mu(1+\alpha)^2r^2\left\{(1+\beta)(\cos.\Theta-\cos.\theta)-\tfrac{1}{2}(\cos.^2\Theta-\cos.^2\theta)\right\} \quad \ldots \quad (21)$$

Assuming then $\Theta=0$, we have $\frac{Y}{r^2}=\mu(1+\alpha)^2\left\{(1+\beta)\sin.\Psi-\tfrac{1}{4}\sin.2\Psi-\tfrac{1}{2}\Psi\right\}$

$$\text{and } \frac{X}{r^2} = \mu (1+\alpha)^2 \left\{ (1+\beta) \text{ vers. } \Psi - \tfrac{1}{2} \sin.^2 \Psi \right\}$$

$$\therefore \frac{Y}{r^2} \sin. \Psi - \frac{X}{r^2} \cos. \Psi = \mu (1+\alpha)^2 \left\{ (1+\beta) \text{ vers. } \Psi - \tfrac{1}{2} \Psi \sin. \Psi \right\} \quad \ldots \ldots \quad (22)$$

Substituting this value in equation (7), making $Y x - X y = 0$, solving that equation in respect to $\frac{P}{r^2}$ and making $\frac{p}{r} = 1 + \lambda$ we have

$$\frac{P}{r^2} = \frac{\{ \frac{1}{2}\alpha^2 + \alpha - \frac{1}{2}\mu (1+\alpha)^2 \} \Psi \sin. \Psi - \{ \alpha + \alpha^2 + \frac{1}{3}\alpha^3 - \mu (1+\alpha)^2 (1+\beta) \} \text{vers. } \Psi}{\lambda + \text{vers. } \Psi} \quad \ldots \ldots \quad (23)$$

If, instead of supposing the pressure of the water to be borne by the extrados, we suppose it to take effect upon the intrados tending to *blow up* the arch, and if β represent the height of the water above the crown of the intrados, we shall obtain precisely the same expressions for X and Y as before, except that r must be substituted for $(1+\alpha) r$, and that both X and Y must be taken *negatively*; in this case therefore $\frac{Y}{r^2} \sin. \Psi - \frac{X}{r^2} \cos. \Psi = -\mu \left\{ (1+\beta) \text{ vers. } \Psi - \frac{1}{2} \Psi \sin. \Psi \right\}$, whence, by substitution in equation (7), and reduction

$$\frac{P}{r^2} = \frac{(\frac{1}{2}\alpha^2 + \alpha + \frac{1}{2}\mu) \Psi \sin. \Psi - \left\{ \alpha + \alpha^2 + \frac{1}{3}\alpha^3 + \mu (1+\beta) \right\} \text{vers. } \Psi}{\lambda + \text{vers. } \Psi} \quad \ldots \ldots \quad (24)$$

Now, by note, page 52, $\dfrac{d\left(\frac{P}{r^2}\right)}{d\Psi} = 0$, differentiating equations (22) and (23) therefore, and reducing, we have

$$\Psi \left\{ \tan. \frac{\Psi}{2} - \lambda \cot. \Psi \right\} + \cos. \Psi + A\lambda - 1 = 0 \quad \ldots \ldots \quad (25)$$

which equation applies to both the cases of the pressure of a fluid upon an arch with equal voussoirs; that in which its pressure is borne by the extrados, and that in which it is borne by the intrados, the constant A representing in the first case the quantity $\frac{\frac{1}{2}\alpha^2+\frac{1}{3}\alpha^3-\mu\left(\frac{1}{2}+\beta\right)(1+\alpha)^2}{\frac{1}{2}\alpha^2+\alpha-\frac{1}{2}\mu(1+\alpha)^2}$ and in the second case $\frac{\frac{1}{2}\alpha^2+\frac{1}{3}\alpha^3+\mu\left(\frac{1}{2}+\beta\right)}{\frac{1}{2}\alpha^2+\alpha+\frac{1}{2}\mu}$. If the line of resistance pass through the summit of the key-stone, λ must be taken $=\alpha$. If it pass along the inferior edge of the key-stone, $\lambda=0$. In this second case, $\tan.\frac{\Psi}{2}\left\{\Psi-\sin.\Psi\right\}=0 \therefore \Psi=0$, so that the point of rupture is at the crown of the arch. For this value of Ψ, (23) and (24) become vanishing fractions, whose values are determined by known methods of the different calculus to be,

When the pressure is on the extrados $\frac{P}{r^2}=\alpha-\frac{1}{3}\alpha^3+\beta\mu(1+\alpha)^2$ . . . . (26)

When the pressure is on the intrados $\frac{P}{r^2}=\alpha-\frac{1}{3}\alpha^3-\beta\mu$ . . . . (27)

The equations given above completely determine the value of P, subject to the first of the two conditions stated in page 13, viz., that the line of resistance passing through a given point in the key-stone determined by a given value of λ, shall have a point of geometrical *contact* with the intrados. It remains now to determine it subject to the second condition, viz., that its point of application P on the key-stone shall be such as to give it the least value which it can receive subject to the first condition. It is evident that, subject to this first condition, every different value of λ will give a different value of Ψ, and that of these values of Ψ that which gives the least value of P, and which corresponds to a *positive* value of λ not greater than α, will be the true angle of rupture, on the hypothesis of a mathematical adjustment of the surfaces of the voussoirs to one another. To determine this minimum value of P, in respect to the variation of Ψ dependent on the variation of λ, or

of p, let it be observed that λ does not enter into equation (8); let that equation, therefore, be differentiated in respect to P and Ψ, and let $\frac{d\,P}{d\,\Psi}$ be assumed $=0$, we shall thence obtain the equation

$$\sec.\,^2\Psi = \frac{3\,\alpha\,(\alpha+2)}{\frac{6\,P}{r^2}+\alpha^2\,(2\,\alpha+3)} \quad \ldots\ldots\ldots\ldots \quad (28)$$

whence observing that $\sin.\,2\,\Psi = \frac{\tan.\,\Psi}{\sec.\,^2\Psi} = \left\{\frac{\frac{6\,P}{r^2}+\alpha^2\,(2\,\alpha+3)}{3\,\alpha\,(\alpha+2)}\right\}\tan.\,\Psi$, we obtain by elimination in equation (8),

$$\sin.\,2\,\Psi - 2\,\Psi = \frac{4\,Y}{\alpha\,(\alpha+2)\,r^2} - 2\,\Theta \quad \ldots\ldots\ldots\ldots \quad (29)$$

from which equation Ψ may be determined. Also by equation (28)

$$\frac{P}{r^2} = \tfrac{1}{6}\left\{3\,\alpha\,(\alpha+2)\cos.\,^2\Psi - \alpha^2\,(2\,\alpha+3)\right\} \quad \ldots\ldots\ldots\ldots \quad (30)$$

and by eliminating sec. Ψ between equations (28) and (29) and reducing

$$\frac{p}{r} = (1+\lambda)\cos.\,\Theta = \frac{r^2}{P}\left\{\sqrt{\alpha\,(\alpha+2)\left\{\frac{2\,P}{r^2}+\alpha^2\,(\tfrac{2}{3}\,\alpha+1)\right\}} - \alpha\,(\tfrac{1}{3}\alpha^2+\alpha+1)\cos.\,\Theta - \frac{Y\,x}{r^3}\right\} \quad \ldots\ldots \quad (31)$$

The value of λ given by this equation determines the actual direction of the line of resistance through the key-stone, on the hypothesis made, only in the case in which it is a *positive* quantity and not greater than α; if it be negative, the line of resistance passes through the bottom of the key-stone, or if it be greater than α, it passes through the top.

It will be observed that the equation (11) determines the angle Ψ of rupture in terms of the load Y, and the horizontal distance x of its centre of gravity from the centre C of the arch, its radius r, and the depth αr of its voussoirs; moreover, that this determination is wholly independent of the angle of the arch, and is the same whether its arc be the half or the third of a circle; also, that if the angle of the semi-arch be less than that given by the above equation as the value of Ψ, there are no points of rupture, such as they have been defined, the line of resistance passing through the springing of the arch and *cutting* the intrados there.

The value of Ψ being known from this equation, P is determined from equation (8), and this value of P being substituted in equation (6), the line of resistance is completely determined; and assigning to θ the value ACB (fig. 25), the corresponding value of ρ gives us the position of the point Q, where the line of resistance intersects the lowest voussoir of the arch or the summit of the pier; moreover, P is evidently equal to the horizontal thrust on the top of the pier, and the vertical pressure upon it is the weight of the arch and load: thus all the elements which determine the equilibrium of the pier (see page 41) are known, and by substituting these in equation (4), page 46, we may determine the greatest height of a pier of given dimensions, or the least dimensions of a pier of a given height, which shall just sustain the proposed arch and its loading.

Thus every element of the theory of the arch and its abutments is involved in the solution in respect to Ψ of

equation (11). Unfortunately this solution presents great analytical difficulties. It may be effected under the form of a series ascending by powers of α, and the two first terms of that series have been given in the paper on the arch, already referred to, in the fifth volume of the Cambridge Philosophical Transactions.

The third and fourth terms of this series assume exceedingly complicated analytical forms. Happily, however, in the failure of any direct means of solution, there are various methods by which the numerical relation of Ψ and Y in equation (11) may be arrived at indirectly. Among them one of the simplest is this :—

Let it be observed that that equation is readily soluble in respect to Y ; instead then of determining the value of Ψ for an assumed value of Y, determine conversely the value of Y for a series of assumed values of Ψ. Knowing the distribution of the load Y, the values of x will be known in respect to these values of Ψ, and thus the values of Y may be numerically determined and may be tabulated ; from such tables may be found by inspection values of Ψ corresponding to given values of Y.

The arch and its loading have here been supposed to be given, and from these it has been sought to determine the conditions of its stability. In the majority of cases, however, the loading, placed within the power of the engineer, has to be *determined ;* the problem being so to load the arch as to be *assured* of its stability. This is a problem of comparatively little difficulty ; we have only to assume a value of Ψ, and from equation (11) to determine the corresponding load. All the circumstances

of the equilibrium of the arch thus loaded will then be known, and we may so determine the dimensions of its piers as to give it any required stability.

The values of Ψ, P, and r, are completely determined by equations (29, 30, 31), and all the circumstances of the equilibrium of the circular arch are thence known, on the hypothesis, there made, of a true mathematical adjustment of the surfaces of the voussoirs to one another; and although this adjustment can (see page 14) have no existence in practice where the voussoirs are put together without cement, yet may it obtain in the cemented arch. The cement, by reason of its yielding qualities when fresh, is made to enter into so intimate a contact with the surfaces of the stones between which it is interposed, that it takes, when dry, in respect to each joint, (abstraction being made of its *adhesive* properties,) the character of an exceedingly thin voussoir, having its surfaces mathematically adjusted to those of the adjacent voussoirs; so that if we imagine, not the adhesive properties of the cement of an arch, but only those which tend to the more uniform diffusion of the pressures through its mass, to enter into the conditions of its equilibrium, these equations embrace the entire theory of the cemented arch. The hypothesis here made probably includes all that can be relied upon in the properties of cement as applied to large structures.

An arch may FALL either by the sinking or the rising of its crown. In the former case, the line of resistance passing through the top of the key-stone is made to cut the extrados beneath the points of rupture; in the latter,

passing through the bottom of the key-stone, it is made to cut the extrados between the points of rupture and the crown.

In the first case, the values of X, Y, and P, being determined as before, and substituted in equation (6), and p being assumed$=(1+\alpha)\ r$, the value of θ, which corresponds to $\rho=(1+\alpha)\ r$, will indicate the point at which the line of resistance cuts the intrados. If this value of θ be less than the angle of the semi-arch, the intersection of the line of resistance with the extrados will take place above the springing, and the arch will fall.

In the second case, in which the crown ascends, let the *maximum* value of ρ be determined from equation (6), p being assumed$=r$; if this value of ρ be greater than R, and the corresponding value of θ less than the angle of rupture, the line of resistance will cut the extrados at the point indicated by this value of θ; the arch will there open at the intrados, and it will fall by the descent of the crown. If the load be collected over a *single* point of the arch, the intersection of the line of resistance with the extrados will take place between this point and the crown; it is that portion only of the line of resistance which lies *between* these points which enters therefore into the discussion. Now if we refer to page 50, it will be apparent that in respect to this portion of the line, the values of X and Y in equations (5) and (6) are to be neglected; the only influence of these quantities being found in the value of P.

Example.—Let a circular arch (fig. 26) of equal

voussoirs have the depth of each voussoir equal to $\frac{1}{10}$th the diameter of its intrados, so that $\alpha = \cdot 2$, and let the load rest upon it by three points A, B, D of its extrados, of which A is at the crown and B, D are each distant from it 45°, and let it be so distributed that $\frac{3}{8}$ths of of it may rest upon each of the points B and D, and the remaining $\frac{1}{4}$ upon A; or let it be so distributed within 60° on either side of the crown as to produce the same effect as though it rested upon these points.

Then assigning one half of the load upon the crown to each semi-arch, and calling x the horizontal distance of the centre of gravity of the load upon either semi-arch from C, it may easily be calculated that $\frac{x}{r} = \frac{3}{4} \sin. 45 = \cdot 5303301$. Hence it appears from equation (15) that no loading can cause the angle of rupture to exceed 65°. Assume it to equal 60°; the amount of the load necessary to produce this angle of rupture, when distributed as above, will then be determined by assuming in equation (11), $\Psi = 60°$, and substituting for $\alpha = \cdot 2$ and for $\frac{x}{r} = \cdot 5303301$. We thus obtain $\frac{Y}{r^2} = \cdot 0138$. Substituting this value of $\frac{Y}{r^2}$, and also the given values of α and Ψ in equation (9), and observing that in this equation $\frac{p}{r}$ is to be taken $= 1 + \alpha$ and $\Theta = 0$, we find $\frac{P}{r^2} = \cdot 11832$. Substituting this value of $\frac{P}{r^2}$ in the equation (6), we have for the final equation to the line of resistance beneath the points B and D

$$\rho = r \frac{\cdot 2426 \text{ vers. } \theta + \cdot 1493}{\cdot 0138 \sin. \theta + \cdot 1183 \cos. \theta + \cdot 22\, \theta \sin. \theta}.$$

If the arc of the arch be a complete semicircle (fig. 26), the value of ρ in this equation corresponding to $\theta = \frac{\pi}{2}$ will determine the point Q, where the line of resistance intersects the abutment; this value is $\rho = 1 \cdot 09\, r$.

If the arc of the arch be the third of a circle (fig. 27), the value of ρ at the abutment is that corresponding to $\theta = \frac{\pi}{3}$; this will be found to be r, as it manifestly ought to be, since the points of rupture are in this case at the springing.

In the first case the volume of the semi-arch and load is represented by the formula

$$r^2 \left\{ (\tfrac{1}{2}\alpha^2 + \alpha) \frac{\pi}{2} + \frac{Y}{r^2} \right\} = \cdot 3594\, r^2,$$

and in the second case by

$$r^2 \left\{ (\tfrac{1}{2}\alpha^2 + \alpha) \frac{\pi}{3} + \frac{Y}{r^2} \right\} = \cdot 2442\, r^2.$$

Thus, supposing the pier to be of the same material as the arch, the volume of its material, which would have a weight equal to the *vertical* pressure upon its summit, would in the first case be $\cdot 3594\, r^2$, and in the second case $\cdot 2442\, r^2$, whilst the *horizontal* pressures P, would in both cases be the same, viz. $\cdot 11832\, r^2$; substituting these values of the vertical and horizontal pressures on the summit of the pier, in equation (4), page 46, and for k writing $K - (\rho - r)$ we have in the first case

$$H = \frac{\cdot 3594\,(2\,K - \cdot 09\,r)\,r^2}{\cdot 11832\,r^2 - 2\,K^2}; \quad \ldots\ldots \quad (32)$$

and in the second case

$$H = \frac{\cdot 4884\,K\,r^2}{\cdot 11832\,r^2 - 2\,K^2}; \quad \ldots\ldots\ldots\ldots \quad (33)$$

where H is the greatest height to which a pier, whose width is 2 K, can be built so as to support the arch.

If $2\,K^2 - \cdot 11832\,r^2 = 0$, or $K = \cdot 243\,r$, then in either case the pier may be built to any height whatever, without being overthrown. In this case the breadth of the pier will be nearly equal to ¼th of the span. The dotted lines in the figure show the dimensions of such a pier.

The height of the pier being *given*, (as is commonly the case,) its breadth, so that the arch may just stand firmly upon it, may readily be determined. As an example, let us suppose the height of the pier to equal the radius of the arch. Solving equations (32) and (33) in respect to K, we shall then obtain in the first case $K = \cdot 1489\,r$, and in the second $K = \cdot 15\,r$.

If the span of each arch be the same, and r_1 and r_2 represent their radii respectively, then $r_1 = r_2 \sin. 60°$; supposing then the height of the pier in the second arch to be the same as that in the first, viz. r_1, then in the second equation we must write for H, $r_2 \sin. 60°$. We shall thus obtain for K the value $\cdot 14\,r_2$.

The piers shown by the dark lines in the figures 26 and 27 are drawn to a scale, and are of such dimensions as just to be sufficient to sustain the arches which rest

upon them, and their loads, both being of a height equal to the radius of the semicircular arch. It will be observed, that in both cases the load $Y = \cdot 0138\, r^2$, being that which corresponds to the supposed angle of rupture 60°, is exceedingly small.

Let us next take the example of a Gothic arch, and let us suppose, as in the last examples, that the angle of rupture is 60°, and that $\alpha = \cdot 2$; but let the load in this case be imagined to be collected wholly over the crown of the arch, so that $\frac{x}{r} = \sin. 30°$. Substituting in equation (11), 30° for Θ, and 60° for Ψ, and $\cdot 2$ for α, and sin. 30° for $\frac{x}{r}$, we shall obtain the value 1·3101 for $\frac{Y}{r^2}$; whence by equation (9) $\frac{P}{r^2} = \cdot 67008$, and this value being substituted, equation (6) gives 1·048 for the value of ρ when $\theta = \frac{\pi}{2}$. We have thus all the data for determining the width of a pier of given height, which will just support the arch. Let the height of the pier be supposed, as before, to equal the radius of the intrados; then since the weight of the semi-arch and its load is $1 \cdot 54\, r^2$, and the horizontal thrust $\cdot 67008\, r^2$, the width 2 K of the pier is found by equation (4) to be $\cdot 422\, r$.

Fig. 28 represents this arch; the square, formed by dotted lines over the crown, shows the dimensions of the load of the same materials as the arch, which will cause the angle of the rupture to become 60°; the piers are of the required width $\cdot 422\, r$, such that when their height is equal to AB, as shown in the figure, and the arch bears

this insistent pressure, they may be on the point of overturning. The great amount of the load Y, which in this arch corresponds to an angle of rupture of 60°, shows that angle to be much less than 60° in the great majority of the cases which are offered in the practice of the Gothic arch.

It is not possible, within the limits necessarily assigned to theoretical investigations in a work like this, to enter further upon the discussion of those questions whose solution is involved in the equations which have been given; these can, after all, become accessible to the *general* reader only when tables shall be formed from them.

The equilibrium of the arch is a particular case of the equilibrium of a system of bodies in contact. The conditions of the equilibrium of such a system will be found discussed under a general form in the APPENDIX.

H. MOSELEY.

King's College,
November 5th, 1839.

APPENDIX

TO

PROFESSOR MOSELEY'S THEORY OF THE ARCH.

GENERAL CONDITIONS OF THE EQUILIBRIUM OF A SYSTEM OF BODIES IN CONTACT.

1. LET a continuous mass, to which are applied certain forces of pressure, be supposed to be intersected by a *plane* whose equation is

$$z = \mathrm{A}x + \mathrm{B}y + \mathrm{C} \quad \ldots\ldots \quad \text{(I.)}$$

Let the sums of the forces impressed upon one of the parts and resolved in directions parallel to three rectangular axes be respectively M_1, M_2, M_3, and the sums of their moments N_1, N_2, N_3.

Let, moreover, the position of the plane be such that these forces are reducible to a single resultant, a condition determined by the equation

$$M_1N_3 + M_2N_2 + M_3N_1 = 0 \quad \ldots\ldots \quad \text{(II.)}$$

The equation to this single resultant will then be

$$\left.\begin{aligned} x &= \frac{M_1}{M_3}z + \frac{N_2}{M_3} \\ y &= \frac{M_2}{M_3}z + \frac{N_1}{M_3} \end{aligned}\right\} \quad \ldots\ldots \quad \text{(III.)}$$

If between the four preceding equations, in which M_1, M_2, M_3, N_1, N_2, N_3, are functions of A, B, C, these three quantities A, B, C be eliminated, there will be obtained an equation in x, y, z, which is that to a surface of which this is the characteristic property,—that it includes all the points of intersection of the

resultant force with its corresponding intersecting plane in every position, which, according to the assumed conditions, this last may be made to take up.

This surface is the **Surface of Resistance.**

If to the preceding conditions there be added this, that in each two consecutive positions of the intersecting plane the corresponding resultants shall intersect, the surface of resistance will resolve itself into a line, which is the **Line of Resistance.**

Differentiating on this hypothesis the equation (III,) in respect to A, B, C, we have

$$\left.\begin{aligned}
&\left\{\frac{d\left(\frac{M_1}{M_3}\right)}{dA}dA+\frac{d\left(\frac{M_1}{M_3}\right)}{dB}dB+\frac{d\left(\frac{M_1}{M_3}\right)}{dC}dC\right\}z+\left\{\frac{d\left(\frac{N_3}{M_3}\right)}{dA}dA+\frac{d\left(\frac{N_2}{M_3}\right)}{dB}dB+\frac{d\left(\frac{N_2}{M_3}\right)}{dC}dC\right\}=0\\
&\left\{\frac{d\left(\frac{M_2}{M_3}\right)}{dA}dA+\frac{d\left(\frac{M_2}{M_3}\right)}{dB}dB+\frac{d\left(\frac{M_2}{M_3}\right)}{dC}dC\right\}z+\left\{\frac{d\left(\frac{N_1}{M_3}\right)}{dA}dA+\frac{d\left(\frac{N_1}{M_3}\right)}{dB}dB+\frac{d\left(\frac{N_1}{M_3}\right)}{dC}dC\right\}=0
\end{aligned}\right\}\ \text{(IV.)}$$

Eliminating z

$$\begin{aligned}
&\left\{\frac{d\left(\frac{M_2}{M_3}\right)}{dA}dA+\frac{d\left(\frac{M_2}{M_3}\right)}{dB}dB+\frac{d\left(\frac{M_2}{M_3}\right)}{dC}dC\right\}\left\{\frac{d\left(\frac{N_2}{M_3}\right)}{dA}dA+\frac{d\left(\frac{N_2}{M_3}\right)}{dB}dB+\frac{d\left(\frac{N_2}{M_3}\right)}{dC}dC\right\}\\
-&\left\{\frac{d\left(\frac{M_1}{M_3}\right)}{dA}dA+\frac{d\left(\frac{M_1}{M_3}\right)}{dB}dB+\frac{d\left(\frac{M_1}{M_3}\right)}{dC}dC\right\}\left\{\frac{d\left(\frac{N_1}{M_3}\right)}{dA}dA+\frac{d\left(\frac{N_1}{M_3}\right)}{dB}dB+\frac{d\left(\frac{N_1}{M_3}\right)}{dC}dC\right\}=0\ .\ .\ \text{(V.)}
\end{aligned}$$

From the elimination of A, B, and C, between the five equations (I, II, III, V), will result the two equations to the LINE OF RESISTANCE; and from the elimination of the same three quantities between the five equations (II, III, IV),* the two equations to the LINE OF PRESSURE.

The inclination ι of the resultant pressure to a perpendicular to the intersecting plane, in any of its positions, may be determined, independently of the line of pressure, from the equation

$$\cos \iota = -\frac{A M_1 + B M_2 + M_3}{\{(A^2 + B^2 + 1)(M_1^2 + M_2^2 + M_3^2)\}^{\frac{1}{2}}}.$$

2. Let the mass be a PRISM whose axis is horizontal (fig. 3), and the forces applied to which are, its weight and certain pressures, P.

The relation of the forces which compose the equilibrium of the whole Prism, will then be the same with that of the forces impressed on any one of its sections perpendicular to the axis.

Let C B D, (fig. 3), represent any one of these sections. Suppose the mass to be intersected in any direction parallel to its axis by a plane, and let $N_1 N_2$ be the intersection of this plane with the section C B of the mass.

And let this intersecting plane in altering its position be supposed to remain always parallel to itself.

Take A z, the axis of z, perpendicular to $N_1 N_2$, and let it make an angle θ with the vertical.

Let $M N_1 = y_1$, $M N_2 = y_2$, $A M = c$, $A K = k$.

* It will be observed that the condition (V) is included in these.

$\therefore \ M_1 = 0, \ M_2 = \Sigma \ P \sin. \Phi - \sin. \Theta \int (y_1 - y_2) \, d\, C, \ M_3 = \Sigma \ P \cos. \Phi + \cos. \Theta \int (y_1 - y_2) \, d\, C,$
$N_1 = \frac{1}{2} \cos. \Theta \int (y_1^2 - y_2^2) \, d\, C + \sin. \Theta \int C (y_1 - y_2) \, d\, C + \Sigma \pm P\, k \cos. \Phi, \quad N_2 = 0, \quad N_3 = 0.$

This hypothesis with regard to the position of the axis of z, and these substitutions being made, all the equations of condition vanish except equation I, the second of equations III, and the second of equations IV. These resolve themselves into the following:—

$$z = C \qquad \qquad (1)$$

$$y = \frac{\left\{\Sigma P \sin. \Phi - \sin. \Theta \int (y_1 - y_2) \, d C\right\} z + \frac{1}{2} \cos. \Theta \int (y_1^2 - y_2^2) \, d C + \sin. \Theta \int C (y_1 - y_2) \, d C + \Sigma \pm P\, k \cos. \Phi}{\Sigma P \cos. \Phi + \cos. \Theta \int (y_1 - y_2) \, d C} \qquad (2)$$

$$y = \frac{z \dfrac{d \Sigma P \sin. \Phi}{d C} d C + \dfrac{d \Sigma \pm P\, k \cos. \Phi}{d C} d C - (y_1 - y_2) \left\{ (z - C) \sin. \Theta - \frac{1}{2} (y_1 + y_2) \cos. \Phi \right\}}{\dfrac{d \Sigma P \cos. \Phi}{d C} d C + (y_1 - y_2) \cos. \Theta} \qquad (3)$$

The equation to the line of resistance is determined by eliminating C between the equations (1) and 2), and that to the line of pressure by eliminating it between (2 and (3).

If the first elimination be made, and it be observed, that

$$- z \int (y_1 - y_2) \, d z + \int z (y_1 - y_2) \, d z = - \iint (y_1 - y_2) \, d z^2,$$

there will be obtained the following general equation to the line of resistance,

$$y = \frac{z \Sigma P \sin. \Phi + \frac{1}{2} \cos. \Theta \int (y_1^2 - y_2^2) \, d z - \sin. \Theta \iint (y_1 - y_2) \, d z^2 + \Sigma \pm P\, k \cos. \Phi}{\Sigma P \cos. \Phi + \cos. \Theta \int (y_1 - y_2) \, d z} \qquad (4)$$

The second elimination is greatly simplified in the case in which P, Φ, k are independent of C. Since in this case, equation (3) gives

$$y-\tfrac{1}{2}(y_1+y_2)+(z-\mathrm{C})\tan.\Theta=0 \quad \ldots\ldots \quad (5)$$

If the intersections be supposed to be made horizontally, (fig. 4,) Θ must be assumed $=0$. If they be made vertically, (fig. 5,) $\Theta=\frac{\pi}{2}$. In the latter case, equation (5) gives $\mathrm{C}=z$.

The elimination of C between (3) and (2) is therefore the same as that between (1) and (2), and the line of pressure in this case coincides with the line of resistance.

3. Let the mass be of a trapezoidal form (fig. 6).

Let A B and C D be inclined to the axis of z at angles α_1, α_2, and assume $\mathrm{CA}=a$; $\therefore\ y_1=a+z\tan.\alpha_1$, $y_2=z\tan.\alpha_2$.

Hence
$$\left.\begin{aligned}\int(y_1^2-y_2^2)\,dz &= az\,(a+z\tan.\alpha_1)+\tfrac{1}{3}z^3(\tan.^2\alpha_1-\tan.^2\alpha_2)\\ \int(y_1-y_2)\,dz &= az+\tfrac{1}{2}z^2(\tan.\alpha_1-\tan.\alpha_2)\\ \iint(y_1-y_2)\,dz^2 &= \tfrac{1}{2}az^2+\tfrac{1}{6}z^3(\tan.\alpha_1-\tan.\alpha_2)\end{aligned}\right\} \quad \ldots\ldots \quad (6)$$

Therefore by substitution in equation (4) we have for the equation to the line of resistance

$$y=\frac{\tfrac{1}{6}z^3\{\tan.\alpha_1-\tan.\alpha_2\}\{\tan.\alpha_1+\tan.\alpha_2-\tan.\Theta\}+\tfrac{1}{2}az^2\{\tan.\alpha_1-\tan.\Theta\}+z\{\sec.\Theta\Sigma\mathrm{P}\sin.\Phi+\tfrac{1}{2}a^2\}+\sec.\Theta\Sigma\pm\mathrm{P}k\cos.\Phi}{\tfrac{1}{2}z^2\{\tan.\alpha_1-\tan.\alpha_2\}+az+\sec.\Theta\,\Sigma\,\mathrm{P}\cos.\Phi} \quad (7)$$

This equation being of three dimensions in z, it follows that for *certain values* of y there are three possible values of z. The curve has therefore a point of contrary flexure, and is somewhat of the form shown in the figure.

A SERIES OF PAPERS

ON THE

FOUNDATIONS OF BRIDGES.

BY T. HUGHES, Civil Engineer.

No. 1.

The foundations of a bridge, consisting properly of the underground work of the piers and abutments, must naturally claim, in a very eminent degree, the attention of the engineer or bridge architect.

The most refined elegance of taste, as applied in the architectural embellishment of the structure—the most scientific arrangement of the arches, and disposition generally of the superior parts of the work—and the most judicious and workmanlike selection, and subsequent combination of the whole materials composing the edifice, are evidently secondary to the grand object of producing certain firm and solid bases, whereon to carry up to any required height the various pedestals of support for the arches of the bridge.

Indeed, the necessity of firmness and solidity in the foundations will be deemed of importance just in pro-

portion to the intended extent and magnificence of th structure they are designed to support.

The distinction made by the celebrated and learne Leon Battista Alberti between the structure abov ground and the foundations of any building is remark ably applicable to the case of bridges. He consider the foundation, not as a part of the structure itsel but as an artificial support on which the latter is to b placed; and justly observes, that if the natural site of building consisted of rock, or other stratum equally har with the material of which foundations are constructed these would be unnecessary, and the building might b commenced and carried up without previous preparatio of the bottom.

In founding bridges to be built over roads and rail ways, it seldom happens that much water is met with and the simple expedient of piling, or using concret where the bottom is unsound, may readily be effecte and will usually ensure the stability of the superior work The difficulty which sometimes will occur in dry an easily accessible cases of foundations will hereafter b discussed; the immediate object to which this paper wi be principally confined being that of elucidating fro practical example the most approved methods of foundin the more difficult and important kinds of bridges, in tended to carry roads of communication across cana and rivers. Although in such bridges the abutmen may occasionally be founded on the dry land, and i some situations where the course of the river can b turned, a similar facility for founding the piers may b

obtained, yet many instances have occurred, and many will, no doubt, occur in future, where bridges must be built over deep and rapid rivers, the course of which can on no account be diverted, and where, consequently, all the difficulties of operating in the midst of water must be contended with and overcome.

Before attempting to classify and describe in detail the various methods at present in use for constructing the foundations of bridges under water, it may be interesting briefly to review some of the older and ruder contrivances of our ancestors. Treating on this subject, an article of great research and ability is given in the Encyclopædia Metropolitana, which describes four methods of laying foundations, by means of caissons, the oldest form of which is no more than a basket of strong construction, such as are made of the pliable boughs or branches of trees, which, being weighted with stones, are lowered to the bottom, and then filled with the same description of material, until raised to within a foot or sixteen inches of the lowest water surface. The application of ingenuity and science to works of this kind naturally led to the substitution of wooden chests, strongly hooped with iron, instead of the original baskets; and these wooden chests, being filled with a sufficient weight of masonry, were sunk to the bed of the river, as practised by Labelye and Mr. Mylne in the construction of Westminster and Blackfriars Bridges. It must not be supposed, however, that the idea of using caissons of a sufficient size to

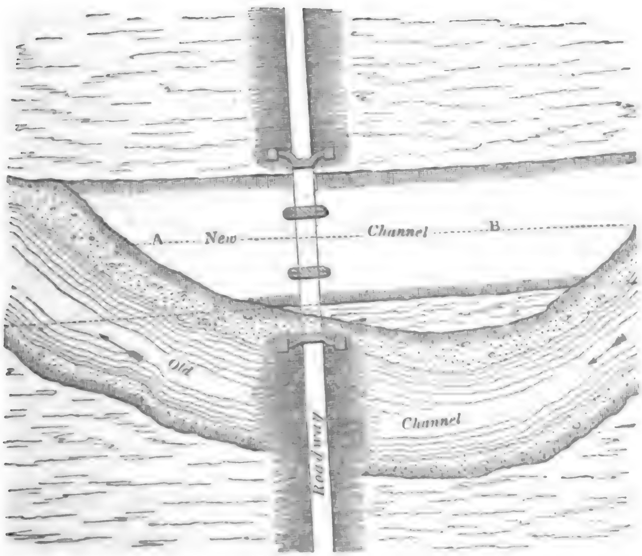
contain a whole pier was put in practice in the earl days of bridge building. The caissons of basket-wor here mentioned were of small dimensions, and severa of them, according to their size, were required to com pose the foundation of a pier or abutment. The firs use also of wooden caissons was limited to those of fiv or six feet square, and it may therefore be ranked as bold and most important advance of modern science t practise the expedient of constructing in a caisson abov ground immense masses of masonry, weighing man hundred tons, and sinking them, still solid and united down to the ground, through a considerable depth o water.

Considering only shallow rivers, it would seem tha the rude caissons used by our ancestors effected tolerabl well the purpose for which they were designed, but i rivers of magnitude, and particularly those where stron currents existed, it was found necessary to construc the piers on dry ground. The most obvious expedien for attaining this end is, in the first instance, to preserv a good and direct approach, leading to and over th bridge, and then to place the site, not over the river but at some convenient distance from it, forming th piers in a range with the general direction of the stream and at right angles with the approach alluded to, the turning the course of the river by a new channel throug the water way of the bridge.

This will at once be understood by referring to th accompanying figure 1, which represents the plan of

bridge to be built in a neck of land, formed by a bend of the river, which must afterwards be diverted from its

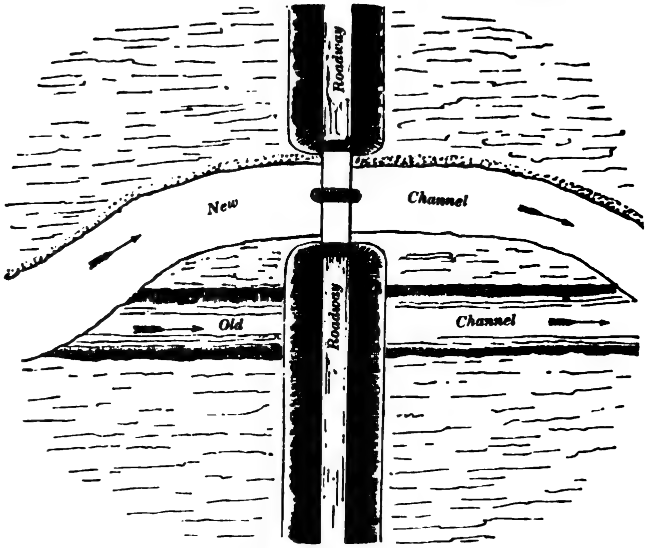
FIG. 1.



course in the direction of the dotted line AB, and the roadway can be readily embanked across the old channel, which will be laid dry. Even when the original course of the river is nearly straight, the expense of fixing the bridge in that situation might be attended with such serious difficulties, as to warrant the engineer in slightly turning the river, and thus have the advantage of building on a firm bottom at a very moderate cost, as compared with the other situation, and so avoid the danger and difficulty to which I allude ; the accompanying sketch, fig. 2, shows the appearance of the diversion ; the bridge to be built on dry land on either side of the old channel.

Such are the circumstances of position in which many

FIG. 2.



important bridges have been constructed, and as the execution of works, under similar circumstances, will in all probability prove of frequent occurrence to modern practitioners, it may be necessary, in a future page, to make some remark on the most practicable method of building the bridge, and effecting the diversion of the river in such situations.

This, it will obviously be seen, involves the necessity, during the period of getting in the foundations, of keeping them clear of water, which might drain into them from the adjacent river; and again, some skill and experience will naturally be required in the management of the river at the time when the new channel is prepared to receive it, otherwise dangerous consequences might arise from the overflow, and breaking down of the banks, or temporary dams or stanks.

All this will be treated of in future, whilst at present we confine ourselves to the kind of foundation adopted by the ancients for large bridges, built in the situation we have been describing.

This brings us to the subject of piles, which, when the bottom was very unsound, were driven down entirely over the site of the proposed piers. The most remarkable difference between the practice of the ancients and the moderns in founding on piles as a foundation, is that the former, instead of evenly cutting off the pile heads, and planking over them with a platform of timber, were accustomed to fill in between them, with a species of coarse concrete, called by the French *bétón*, and which, being brought to a level surface, formed the bed on which the first course of stone was placed.

We believe that since some recent publications on the subject of limes and concrete, particularly that very excellent one by Colonel Pasley on calcareous mortars and cements, and another by Mr. Godwin on concrete, no apology is necessary for what some years since would have been pronounced the anachronism of ascribing the use of concrete to the ancients, and especially to the Romans. Both the authors above mentioned concur in admitting that the use of a concrete mixture, composed of lime and coarse gravel, is by no means an exclusively modern practice.

The piles for foundations of old bridges were distributed much in the same way as for modern structures, namely, three or four feet apart, and the extent to which

they were driven was regulated, as of course it ought to have been, by the nature of the ground bottom.

Another method of using piles, besides that of establishing a foundation for the masonry, was much practised by the ancients for building in water. This method is called by the French *encaissement*, and is still practised along the shores of the Mediterranean. The following is the system pursued. Main piles are driven in, with sheet-piling between them, secured and bound round with waling, as in the modern coffer-dam, but it must be understood that for the purpose of encaissement, only one row of piles is to be driven all round the space of the pier or abutment to be founded, and not a double row of piles to be filled with clay as in a coffer-dam. When the space to be occupied by the foundation has been entirely enclosed by this wooden encaissement, and the compressible material within the enclosure has been taken out, a mass of concrete or dry rubble stones is thrown into it, and the foundation is formed in this manner until the work reaches the level of the water. Foundations within encaissements should be allowed some time to settle, and become solid before the dressed masonry is laid in courses upon them.

In this manner many celebrated works were founded on the Continent, and the system was particularly approved of by Belidor, who much preferred it to that of constructing foundations entirely of dressed stones. Another great argument in favour of this system arises from the saving of coffer-dams, which are always found to be very tedious and expensive works.

As connected with the use of concrete during late years in the foundations of bridges, Mr. Peter Semple, architect of a bridge over the Liffey at Dublin, seems entitled to considerable credit, if not for the re-invention of this substance, at least for setting the example of using it on an extensive scale.

This author gives a design, which bears a precise resemblance to the form of encaissement above described. In a river six feet deep, for example, he proposes to drive sheeting piles, about ten feet in length, to a depth of four feet into the ground all round the site of each pier, and to fill the space or coffer thus formed with a bed of concrete, six feet in depth.

The top of the concrete being level with the surface of summer low water mark, he proposes thereupon to commence the masonry, and carry it up to springing height clear of the water.

Before concluding this brief glance at the practice of the ancients, it may be necessary to notice, that on examining the foundations of old bridges, particularly in this country, they are all found to be extremely massive, and the piers were even carried up above water, of a thickness quite incommensurate with the necessity of the case. On inspecting the masonry of their foundations, however, it is found that no very great attention has been paid to the regularity of courses, or to the perfection of beds and joints. Some of the strongest specimens of ancient masonry existing in this country consist of a kind of building little superior to rubble walling, with this most important qualification, that the

mortar is always of an excellent description, and in most cases, by no means inferior in hardness and cohesiveness to the stone itself.

It is no uncommon thing to witness, for instance, the face of old buildings, where the masonry has been indented and sensibly decomposed by the varied action of air, water, and frost, while the horizontal bands or joints of mortar project sensibly beyond the stone-work, clearly indicating that a greater resistance has here been offered than by the stone itself. The mortar of the ancients usually contained a great number of small stones or pebbles, some of them equal in size to a pigeon's egg, and was altogether of a much coarser description than that used in the present day. Another point of importance in the bridge architecture of the ancients is deserving of attention. For the most part, the bold designs of modern engineers, who devote their talents to the object of spanning the most majestic rivers with one vast arch, were preceded by structures, consisting of a long low series of culverts, hardly deserving the name of arches, with intervening piers, often of greater thickness than the span of the arches they were built to support. Bridges of this kind are worthy of no higher appellation than very thick walls, or embankments, perforated by a multitude of small openings, and, as compared with the structures of modern days, they are utterly contemptible and insignificant. But in founding the piers of such bridges a great deal of the difficulty attendant on the modern works was entirely avoided. We know that if a given weight be distributed among a great many points

of support, the pressure upon each point is proportionably diminished; and hence the twenty or thirty piers, which were sometimes built in the old bridges, had each to support very little more than its own weight.

It is easy from this consideration to discover one reason why the precaution of piling was so commonly dispensed with, although it would probably be erroneous to attribute it entirely to this circumstance. The ancients, in their bridges, throughout the whole structure, substituted quantity for quality, that is to say, huge masses of rough undressed masonry, or rubble, instead of the firm, compact, and elegant piers of modern bridges, which, above the bed of the river at all events, are invariably built with the most durable stone, of well-squared dressed ashlar fronts and suitable filling within.

The charge of clumsiness, however, conveys no reproach upon the judgment of an engineer, if it be confined to the work sunk into the ground under water, particularly as in such a situation it presents no obstruction to the free course of the river.

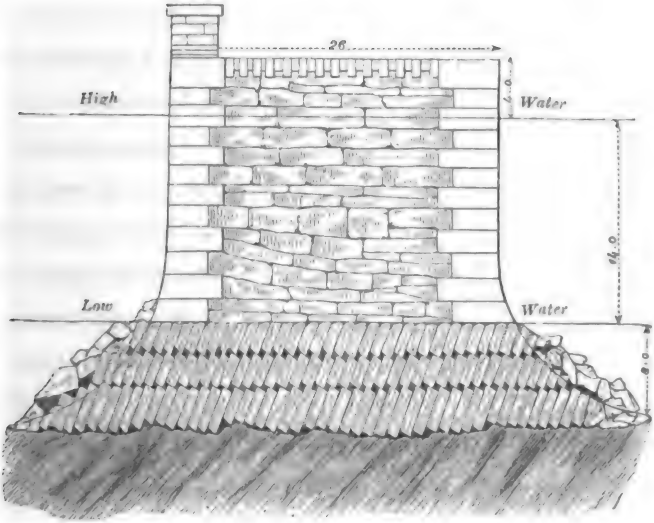
Indeed, it may often be most judicious, because most economical, to construct large and massive bases for the piers of a bridge to be raised upon, instead of building up regularly dressed courses of masonry from the very bottom, within a coffer-dam.

As this practice has been very successfully adopted in some projecting sea piers of modern construction, as well as of great magnitude, it may not be uninteresting

to describe them as practical examples, which may be safely followed under corresponding circumstances. The first work of the kind I shall describe was projected by Mr. Telford, and executed under the superintendence of Mr. David Henry, at Ardrossan Harbour, in Ayrshire, N. B.; and as the mass of stones used in the foundation was there set in tolerably regular order under water, without the aid of coffer-dam, or caisson of any kind, there can be no doubt of the same system being equally practicable in many cases of bridge foundations.

The stones at Ardrossan were of very large superficial dimensions, varying from six to ten feet long, and three to five feet wide; they were first held fast by an implement, technically called nippers or devil's claws, and were then lowered by a crane through a depth of six or eight feet of water on to a hard and solid foundation. The blocks were placed end to end, the position of the last stone lowered being found by probing with a slight iron rod; and as soon as each stone was in its place longitudinally, the claws were disengaged, and the stone allowed to rest upon the course below, as seen in fig. 3. The courses were continued entirely through the whole thickness of the pier; and when a sufficient number had been laid to bring the work up to the height of low water spring tides, the whole breadth was levelled, and all the unequal projections chipped off, in order to prepare a bed for the first course of dressed masonry. The work then proceeded in the regular manner, consisting of alternate headers and stretchers of properly squared

Fig. 3.



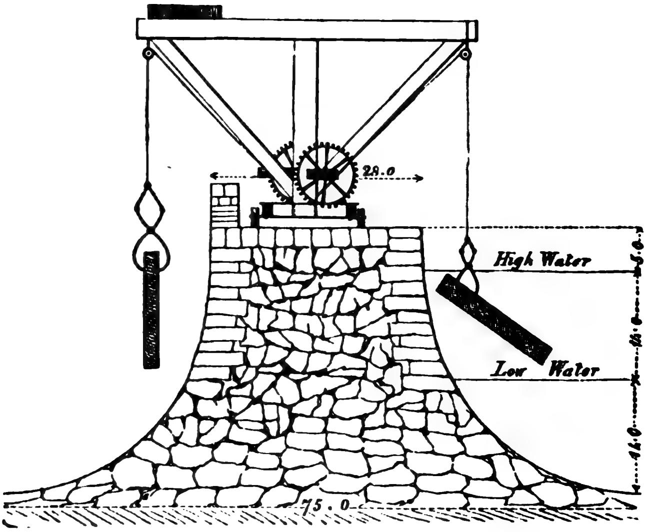
ashlar in front, with dry stone hearting of squared scapple dressed rubble inside, and in this way was carried up to the full height required.

When the writer visited this work, in the year 1818, it had been advanced a considerable distance into the sea; and although parts of it had been exposed to some very heavy storms, neither flaw nor settlement could be discovered in any part of this excellent piece of dry-built masonry.

From an account of some foundations similar to that described above in the recently published life of Mr. Telford, it may be seen that the practice has been much more extensively adopted, and a far bolder attempt carried out by Mr. Gibb, of Aberdeen, than the one acted upon in the other work at Ardrossan. The pier at Aberdeen is extended into the sea, with a breadth at

the base of seventy-five feet, the bottom consisting entirely of irregularly shaped masses of stone, which having been conveyed to the spot in boats, were tumbled in by chance to the depth of ten or twelve feet. In the drawings composing the Atlas, which accompanies the life of Mr. Telford, the low water mark is shown about fourteen feet above the bottom, and in the narrative of this work by Mr. Gibb, he states, that the bottom under the foundation is nothing better than loose sand and gravel, and that the front ashlar commences at about one foot under low water mark, and is carried up to the top of the pier, which the drawing shows to be about thirty-three feet in height from the bottom to the top. The rise of the tide is shown to be fourteen feet, the breadth of the pier twenty-eight feet, the sides carried up with a slope inwards. Fig. 4. describes the method adopted by Mr. Gibb.

PIER AT ABERDEEN. FIG. 4.



The last description of foundation which we shall describe as practicable in many situations, where the necessary funds cannot be obtained for raising the piers of a bridge within a coffer-dam, or on a caisson bottom, was practised at Inverness, under the superintendence of the writer, agreeably to instructions given by Mr. Telford. In order that the nature of this work may be understood, it must be explained that the old harbour of Inverness is situate on a part of the river Ness, about a quarter of a mile from its entrance into the firth or arm of the sea, with which it communicates. The river has a considerable fall from the old harbour towards the sea, and the tide does not rise within several feet of the same height at the former place, as it does lower down. The consequence was, that the larger description of vessels which frequented the port could navigate the river only at high water of spring tides; and as the trade increased in common with that of many other ports, during the late war, this inconvenience was most sensibly felt.

The town funds were inadequate to defray the expense of deepening the river by bringing up the dead level from the sea, and afterwards building from the bottom a suitable breast work for unloading at the harbour. The expedient eventually decided upon was that of building a pier or landing wharf at some distance down the river, at a place where the water was already tolerably deep, and at which large vessels, passing up the river, might discharge a part of their cargo, and then, after being thus lightened, and rendered capable of floating in a less depth of water, pass up with the remainder of their

cargo into the more convenient accommodation afforded by the harbour.

The usual and modern practice of founding works of this kind is by coffer-dam, but the low state of the funds applicable to the work rendered it impossible to carry into effect so expensive an operation. In this extremity, Mr. Telford was consulted, and he, considering all the circumstances of the case, and particularly the great necessity for practising economy, gave instructions for a mode of putting in the foundations, which it may be useful to describe in detail. At the site fixed upon for the intended pier, the depth of water, at the lowest spring tides, was never less than four feet, and at ordinary low water five or six feet; the bottom a very hard gravel, united with clay. The whole length of the breast work was about one hundred and sixty feet, and throughout this distance the bottom was dredged out, to the width of eight feet, and depth of two feet, to receive the masonry.

A simple system of piling was however driven previous to founding the masonry. The piling consisted of two bearing piles, twelve feet long, and eight inches diameter, driven down at intervals of twenty feet; and across the heads of these piles, and level with low water mark, cross pieces of elm planking twelve feet long, three inches thick, and one foot wide, were fastened with trenails. On the top of these were laid longitudinal half timbers, one foot wide, and six inches deep, secured to the cross pieces and bearing piles by rag bolts, driven into each pile head.

The accompanying sketches, figs. 5 and 6, will amply illustrate the forms and disposition of the timber work in the foundation. In addition to the bearing piles,

Fig. 5.

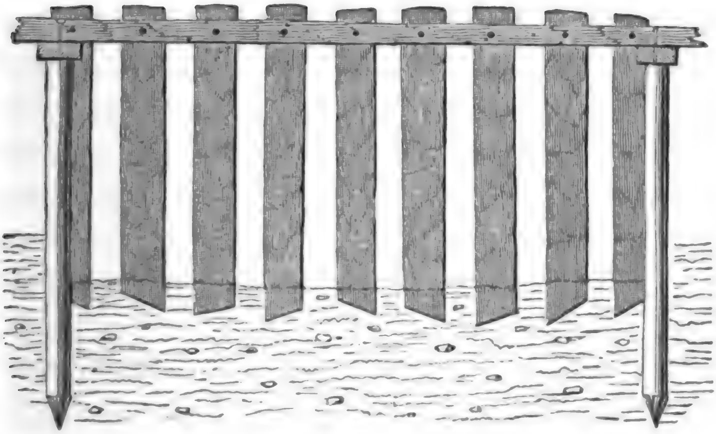
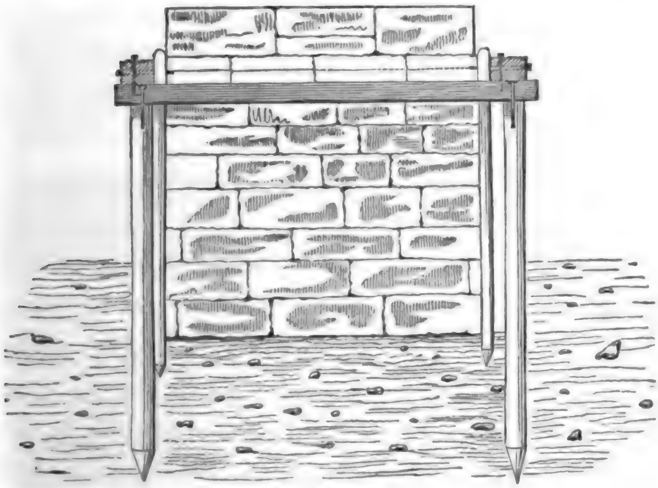


Fig. 6.



a row of timber slabs, of inferior quality, was also driven down a few inches into the bottom, at intervals of about ten or twelve inches; these had a spike driven through

them, near their heads, and into the longitudinal logs of half timbers; they were merely to answer the purpose of guide timbers, to set the stones by, and to determine the guage or breadth of the work, and were afterwards removed.

The bottom on which the pier was to be founded being now made as level as possible by means of dredging with the common bag and spoon apparatus, the stones were brought to the place in boats, and lowered by a crane, in such a way that as soon as each stone was placed in its proper position the *lewis* could be withdrawn without difficulty.

This will be understood on referring to fig. 7, which represents the lewis, fixed in a stone, ready prepared for being lowered through the water into the foundation. The lewis consisted of two pieces of iron B and D, and in order to use it a part of the stone must be cut out, sufficiently wide at top to receive the base of the part B, the base of the opening of the stone being equal to the united width of D and B; A is the chain suspended from the arm of the crane,[1] and E a small rope or string, of which the

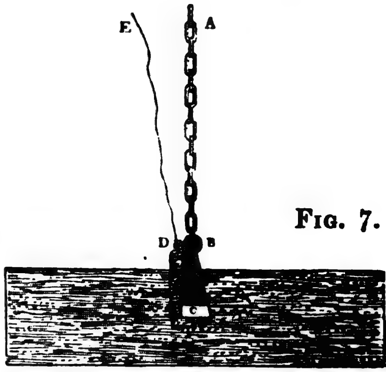


FIG. 7.

[1] It is quite evident that by any other mode of suspending the stones excepting that of the lewis, which could be disengaged under water even an approximation to a close joint could never have been effected in the situation now described.

end is kept above water, to pull out the rectangular part D of the lewis.

It is easy to see the method of using this instrument: the piece B is first inserted, and D is then put in to secure it, when it is evident that the heavier the stone may be, provided it be strong enough, the more securely will it be held by the lewis when suspended from the crane. Conceive the stone now to have been lowered through the water, and carefully laid in its proper place in the foundation; the chain from the barrel of the crane is then loosened, and the part B of the lewis being slightly knocked with an iron rod from above, is easily made to drop down into the vacant space C. It is evident that the fastening piece D will then be loose, because between this and B there is a space left equal to the difference between the base of B, and the base of the opening in the stone. D may therefore be drawn out by means of the string E, and B will readily follow on pulling the chain A, and the lewis is again ready to be inserted in another stone.

All the front stones of the foundation were laid with a lewis of this kind, as well as the backing of squared stones, which were previously scapple-dressed at the quarry. The whole of the stones in any one course, for the length of the pier, were laid of equal thicknesses; they ranged from four to seven feet long, and from three to four feet wide. As soon as one course was complete another was laid, and the length of each stone being marked on the longitudinal beams above the piling, it was easy to set them so as to break bond, and the whole

process of thus building under water was effected with the utmost regularity, and with less difficulty than could have been anticipated by the most sanguine advocates of the plan.

When all the building was carried up as high as the surface of the lowest water mark of a spring tide, any irregularity on the top was taken off, and the whole surface carefully levelled, and on it the ashlar masonry was commenced and carried up with a vertical batter. This work consisted of stones with picked fronts and chisel-draughts round the edges, the ends, beds, and face, properly squared. The backing was of good common rubble, and the whole being raised to three feet above the highest spring tides, was finished off with a heavy coping, properly dowelled, cramped, and secured with lead.

This work, from its situation, is called the Thorn Bush Pier; the date of its construction was 1815, and up to the present time no appearance of failure or imperfection has been observed.

Another proof is thus afforded that foundations of magnitude and importance may be laid under water without resorting either to coffer-dams or caissons. It is important, however, that the engineer, before determining on adopting a plan of this bold and somewhat hazardous nature, should be well satisfied of the firmness and solidity of the bottom on which his work is to be founded. Very hard gravel, some of the clays, and rock, are excellent as foundations; but these, amongst other kinds, will be particularly discussed, when, in a future paper, we shall treat of the probable causes of failure

of settlement, and giving way of the piers and abutments of bridges.

No. 2.

When an engineer decides on founding the piers of a bridge within a coffer-dam, it has been a very general practice to leave the form and construction of the dam entirely to the contractor, on whom rests the responsibility of devising the proper means for keeping out the water while such a foundation is being constructed, as may have been described in the specification, or afterwards directed by the engineer, when he becomes fully acquainted with the nature of the ground to be built upon.

This practice of leaving undefined the mode of constructing the coffer-dam, and thus relying upon the contractor's judgment and experience in all the important details relating to the length and scantling of the piling, the extent of driving, and the thickness of the clay puddle between the rows, has been followed, we believe, almost universally by the railway engineers in their specifications for bridges, where it was likely that a coffer-dam would be required.

Hence, the young engineer must look to other sources than the ordinary modern practice of engineering for information on the subject of coffer-dams, and probably to no work in this country, or indeed in Europe, can he turn with more certainty of deriving the best practical knowledge than to those excellent dams constructed by Mr. Thomas Rhodes at the St. Katherine's Dock, under the direction of Mr. Telford; and those more recently

constructed under the superintendence of Mr. Walker the one at Blackfriars Bridge, rendered necessary during the repair of the foundations, and the other at the site of the New Houses of Parliament at Westminster.

Mr. Walker, who, from his position as President of the Institution of Civil Engineers, and as the successor of Mr. Telford in that honourable station, may be considered the official head of his profession in this country has not thought it beneath his dignity to furnish the contractor with drawings, and a detailed specification of the form and entire construction of the coffer-dam On these, and those constructed by Mr. Rhodes, the young engineer may safely depend in all similar works with which he may be engaged. We shall, however, in our next, give instructions for constructing a dam to stand against forty feet of water.

The practice of employing caissons appears, during late years, gradually to have been given up, and few modern examples of their use can be found; Mr. Walker we believe, used them very successfully at the Vauxhall Bridge. We are not aware that Mr. Telford, in all his practice, ever employed one; his preference, in the case of foundations in deep water which could not be diverted being decidedly given to the system of enclosing the site within a coffer-dam. The best account of caissons particularly those used by Mr. Mylne at Blackfriars Bridge, will probably be found in the article before referred to in the Encyclopædia Metropolitana.

In many tide rivers, where the depth at low water is not considerable, the foundations have been constructed

without coffer-dams, the men working only during certain intervals, immediately before and after flood or low water. This plan of proceeding cannot very successfully be adopted, however, unless the bed of the river be of rock, or other safe material for placing the foundations upon. The instance already given of an extensive harbour wall being founded under water, even where it was necessary to excavate the bed of the river under the site of the wall, deep enough for the foundation to rest upon, is an exception to the usual plan of coffer-damming, but might with great advantage be practised at any time in similar situations, where the depth of water does not exceed seven or eight feet. The method of encaissement filled up with concrete, as described in a former paper, affords great facilities for establishing a foundation in any depth of water not exceeding ten feet.

Probably the most difficult kind of foundation which the engineer can have to contend with is to be found in the flats of Norfolk, and in the extensive fens of Lincolnshire and Cambridgeshire. Throughout these districts the alluvial deposit is of great depth, and the flow of the rivers extremely sluggish in their progress towards the sea; circumstances which have induced the formation of strata which are remarkably treacherous and unsafe as foundations. The strata consist of soft and irregular deposits of fine sand, gravel, and mud, alternating with beds of peat and bog, and occasionally mixed with portions of clay. Sometimes these varieties of soil occur in patches; for example, seven or eight feet

square of gravel, surrounded perhaps by peat, or instead of the gravel fine sand is often found in a similar situation, while perhaps, close at hand, layers of peat have been formed in tolerable order; above them sand may occur, succeeded by gravel, and afterwards peat again. At other places, perhaps also within a short distance, the soil will consist of gravel, peat, and sand, all mixed up together, sometimes to the depth of thirty or forty feet before a foundation of clay or firm gravel can be met with. Under these circumstances, if the depth of water be considerable, and the structure heavy, the engineer's difficulty will be far from trifling, unless indeed he can command an unlimited purse, in which case it would be paying a poor compliment to engineering science to dwell upon the *difficulty* of executing any work. This word, in the vocabulary of an engineer, is only with propriety applied to the task of constructing his works within reasonable limits of expense, and he commits a libel on the judgment and talents of the profession who talks of the difficulty experienced by an engineer whose expenditure has been equally unlimited with his command of capital.

We shall now attempt an outline of the plan which an engineer should follow in preparing his foundations, where the ground is of the nature we have been describing. Suppose the bridge to be built over a river with five feet depth of water at the lowest summer floods, that the breadth of the waterway is great, and that no possibility exists of laying dry the bed of the river by turning its course: in such a situation it is

certain no method can be so good as that of driving a coffer-dam all round the space to be occupied by the piers, as well as the abutments. The depth of water being five feet, and of the unsound bottom say twenty-five feet, the piles can in no case be less than forty-five feet long, and driven into the solid ground as far as they will go, which may probably be from eight to ten feet. For such a depth of water, a double dam with three rows of piles will be necessary. The coffers between the rows of piles should be six or seven feet apart, and filled with a retentive clay puddle: the next operation is to get out all the water and soft material from within the coffer-dam. To effect this, various contrivances will present themselves; among which, in almost any case, a steam engine must be in requisition, for pumping out and clearing the space of water, and for keeping it dry during the progress of the building. The earth may be drawn from within the coffer-dam by means of windlasses, to be erected on a temporary stage or platform placed across the dam; and it will be found an excellent expedient to use buckets provided with bottoms which will open when they are drawn up by the windlasses, and thus discharge their contents into barges placed alongside the coffer-dam.

In this way the space to receive the piers and abutments may be expeditiously taken out, and as soon as the soft material has been removed, should the bottom be found to consist of hard gravel or clay, nothing more will be necessary than to commence the building after sinking into it about two feet. It is always good prac-

tice to lay broad and large bedded stones on the found tions, in order to give as much base, with as few join as possible, for the superstructure to rest upon, wheth it be intended to build the piers with brick or stone : is very important, if practicable, to obtain stones of lar superficies for the first and second courses of t building. The foundations both of piers and abutment when of such a depth as we have been describing, shou be carried up with offsets, so that each course for t first four feet in height from the bottom should proje all round at least six inches beyond the course imm diately above. The offsets may then be discontinue and the building carried on according to the particul form of the design. If, after excavating within t coffer-dam to the depth above described, the botto should still appear doubtful and unsound, a number piles should be driven all over the space to be occupi by the building, and extending about a foot beyond it every direction.

The piles may be eight or nine inches square, round timber of this diameter, and should be driven rows from two to three feet apart, as far as they ca be forced into the solid ground, that is, until each pi will not sink more than a quarter of an inch wi twenty blows from a ram of fifteen hundred poun weight or thereabouts. Some practical men recommen that the piles should be driven first in the centre, an last at the ends and sides, alleging as a reason, that b adopting an opposite practice the ground becomes solid as to prevent the piles from sinking in the centr

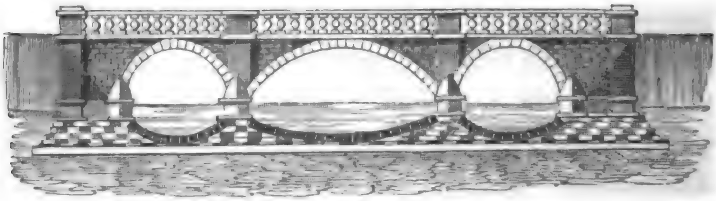
the writer however prefers the method of driving the outer piles first, and in this way consolidating the ground, with as little delay and cost as possible. It is true, by driving from the outside, and closing in towards the centre, that the last piles are prevented from being forced in so deep, but this only renders it necessary to use shorter piles, and the foundation thus becomes quite as solid as if the ground were forced outwards from the centre, and long piles driven in over the whole space. With respect to the description of timber which should be used for these piles, provided it be sound and fresh, the particular kind is of little consequence. Mr. Telford frequently specified Scotch fir; and the inferior kinds of timber, such as beech, elm, and birch, will answer very well, as they stand the driving better than fir, which, if not hooped, is apt to splinter, or the head become besomy and brushy, a circumstance which adds to the difficulty of driving.

As soon as the piles have all been driven, their heads should be cut off quite level, and about a foot in depth being excavated between them, the space up to the level of the pile heads should be filled with broken stones, grouted with good lime and sharp sand. At this stage, a platform of oak, beech, or elm plank, from four to six inches in thickness, should be laid across the pile heads, and secured to them either with spikes, bolts, or trenails of hard wood. This first covering of planking is usually succeeded by another of equal thickness laid across, the whole closely jointed; and on this upper platform the building with brick or stone may safely be commenced.

The same practice of laying the masonry with offsets as before described should be observed here; and although with piling and a planked bottom there does not exist the same necessity for building the first and second courses with large stones, yet the practice is invariably good, and ought always to be followed where such stones can be obtained without a great increase of expense.

Could any plan have been adopted for turning the course of the water, so as to lay dry the bed of the river, an engineer would be able, in the situation we have been describing, to build a bridge at much less cost than by the expensive means of a coffer-dam and piling. Where the river could be turned, the plan we should propose to give stability to the unsound bottom would be to cover it entirely over with cross sleepers of Memel logs, and on these to lay a covering of planks, closely jointed, while further security might be obtained by introducing inverted arches above the planking, between the piers, and extending under each of the abutments, as shown in fig. 8. It will however not be

FIG. 8.



necessary to introduce a platform of timber to support the inverts, unless the ground be too soft to construct them upon it, which however is often the case in situa-

tions similar to those we are describing; and in some cases the platform, without the inverted arches, may be sufficient of itself to carry the bridge: the determination however of introducing or dispensing with these must be left to the decision of the engineer when the lowest bottom can be seen; and all we can do at present is to point out the practice where difficulties exist.

It might be objected with reason to the above method, that the great weight of a bridge on such a foundation would cause it to sink to some extent, and if the bottom were very soft the settlement would no doubt be considerable. From this, however, there would be no danger of fracture, as the settlement would be gradual and uniform, and where the inverted arches are introduced they would most effectually prevent the piers and abutments being moved from their position. In support of this mode of foundation we may cite Mr. Rennie's example in dealing with a similar stratification, while building the Albion Mills close to Blackfriars Bridge. To avoid the expense of placing the foundations at a great depth, and to prevent settlement in the walls, he adopted the expedient of forming inverted arches over the whole space on which the building was to stand. To support the inverts where the ground was softest, some piles were driven, and courses of very large flat stones laid under the walls, while the inverted arches joining into these prevented any unequal settlement. Before quitting the consideration of unsafe ground, such as we have been describing, we must not omit to mention the use of concrete as an excellent

substitute for piling in every situation where this latte would otherwise be necessary to secure a solid foun dation.

The expense of laying a foundation of concrete fron four to six feet in depth, and extending about two fee round the space to be occupied by the building, ought i all such cases to be considered, and compared with th expense of piling, which will probably never be foun more safe than a body of concrete, unless the botton be so soft as to allow the concrete to sink into it, an thus entirely cease to support the building. For it durable and almost imperishable nature, concrete canno be too much esteemed, and besides being quite as safe and perhaps more durable than piling, its cost will i most districts be found much less. Where the piers ar to be built either of brick or stone, a great saving wi be effected by the use of concrete underground, as th expense of this substance will in no case exceed one third the price of any description of brick work, and i some cases it may not cost more than a sixth. I may be thought by some that the description we hav given is exaggerated as to the very unfavourable natur of the stratification in parts of the Crag district of Nor folk and the neighbouring counties; and in order t guard against this notion it may be mentioned, that i directing the works of the North Walsham and Dilhar Canal in Norfolk, the writer had occasion to build a loc in a situation where, after sinking the lock pit dow to the necessary depth for laying the sills at the lowe gates, the bottom was so soft, that a couple of mer

without much labour, worked down an iron bar an inch square to a depth of twenty-eight feet. The bottom was then considered firm, but the expense of excavating to so great a depth through such a soft stratification, surrounded also by a mill-dam, was considered too great for the finances of the Company.

The following description of this work may be interesting, as an example of founding on a timber platform under circumstances of some difficulty.

The idea of introducing a platform of timber, supported by piles driven through the unsound strata to the solid ground, was also abandoned on account of the expense. The bottom was composed of peat or bog interspersed with sand holes, and some pits of fine gravel, which admitted water in all directions; under these circumstances the plan of foundation acted upon was that of laying transverse sleepers of Memel timber across the whole breadth of the lock, and under the side walls and counterforts. The clear width of the lock being fourteen feet, the side walls four feet six inches, and the counterforts four feet, it was necessary that these sleepers should be thirty-two feet long; they were one foot wide and six inches thick, laid at intervals of three feet apart, and extended from under the fore bay to twenty feet beyond the main sill of the lower gates. These sleepers were covered with three-inch planks, placed close together, forming a solid platform of sufficient size to support the whole lock, chamber, side walls, and counterforts, with an extension twenty-four feet further than the gates, in order to prevent the

water from undermining the wing walls when the sluices were opened in the lower gates. Sheeting piles were driven down at the front side of the mitre sills, another row at the front of the main sill, and a third row at the extremity of the platform.

These piles were four-inch beech planks, tongued and grooved, driven by a ringing engine, with a monkey weighing eight hundred pounds, and it is remarkable, notwithstanding the ease with which an iron bar was worked down to a depth of twenty-eight feet, that none of the piles could be forced down more than fifteen feet, and many of them were cut off at nine or ten feet. This difficulty was experienced in consequence of the quicksand adhering so firmly to the sides of the piles as to prevent them alike from sinking, and, when once driven, from being raised by any force that could be applied; so that if the workmen at any time after having driven a pile six or seven feet ceased their blows for only a few seconds, they were quite unable either to raise that pile out of the ground, or to drive it further in.

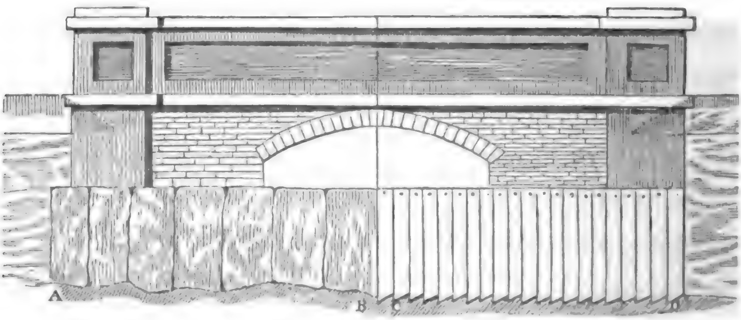
The uncertainty as to the nature of the ground in which piling is to be used has naturally induced engineers to specify indefinitely the length of the piles, because this can only be judiciously determined during the actual driving; and hence it has become a common practice to specify that the piles are to be driven down to solid ground, or otherwise it may be specified, as in the example we have given, that they are to be driven till fifty blows with a monkey of about eight hundred pounds weight will drive them no more than a quarter

of an inch into the ground. Piling is either paid for to the contractor entirely as an extra work, or he is directed to estimate for a specified amount of piling; and then he undertakes by his contract to execute either an additional or diminished quantity, as may be required by the engineer. The Company in the one case allow the value of the additional quantity, and in the other case deduct from the amount of his contract the value of the diminished quantity, if any. Mr. Telford in his practice as an engineer was exceedingly cautious, and never allowed any but his most experienced and confidential assistants to have any thing to do with exploring the foundations of any buildings he was about to erect. This scrutiny into the qualifications of those employed about the foundations extended to the subordinate overseers, and even to the workmen, insomuch that men whose general habits had before passed unnoticed, and whose characters had never been inquired into, did not escape Mr. Telford's observations when set to work in operations connected with the foundations. He was accustomed to examine men so employed whom he thought unsteady, and, if necessary, would reprimand the overseers for employing such men about the foundations in any capacity. It is evident from these precautions that Mr. Telford was well convinced how dangerous it was even to receive a report of the strata from men of careless habits or inefficient knowledge, and that he also knew the consequences which might follow from careless pile-driving, and, in short, from the absence

of proper care in all the operations connected with the commencement of an important structure.

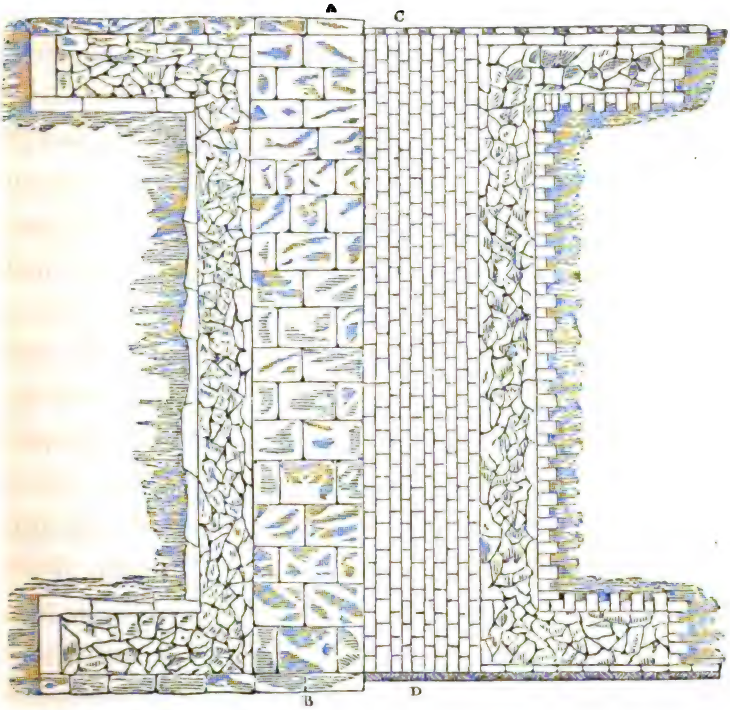
When the site of the foundations, on being explored, was found unsafe, and the expense of driving piles through a very deep bed of loose material would be considerable, Mr. Telford frequently laid an inverted arch between the piers and abutments, as already recommended and shown in fig. 8. At other places, where the ground was more solid, but still doubtful, a pavement of broad stones was placed over the whole bottom, extending also under the abutments and wing walls. These also were tied across the whole bottom by a row of the same description of broad-bedded stones placed on edge, and this kind of pitching was often continued to the extent of the wing walls, and then tied across by a row of the same stones, scabble-dressed, rough-squared, jointed, and set on edge; this plan is described in figs. 9 and 10: that half marked

FIG. 9.



AB in the plan and elevation is in reference to the practice we are now describing. Sometimes these stones could be procured in lengths of six or seven feet, from

Fig. 10.



three to five feet broad, and from six to nine inches in thickness; and when stones of these dimensions could be procured Mr. Telford invariably adopted them, as a covering, in preference to timber. In many places, where large stones could not so readily be obtained, he directed a pitched bottom to be formed with stones of about two feet superficial area, and not less than a foot deep. These were pitched endways, were close jointed throughout their whole depth, and always set with considerable care, the practice being never to permit them to be set on sand, or any material that could afterwards be carried away by the water. This pitching was usually

secured at the end of the abutments and wing walls by
longitudinal sleeper placed along the outside, with shee
ing piles driven down in front to a depth of from six t
ten feet, according to the nature of the bottom he wa
founding upon ;—that part of figs. 9 and 10 marked C
shows this method. Amongst the variety of expedien
adopted by Mr. Telford in his foundations of the Highlan
road-bridges, the one eventually decided on was usual
undetermined until he became acquainted with the preci
nature of the stratification ; and even where he specifi
particular depths and forms of foundation, he alwa
reserved the power of altering these in any way whic
after circumstances might lead him to decide upon.
his general specifications for Highland roads and bridg
he directs, " Where there is no rock the foundation is
be sunk two feet below the bed of the river, and if t
ground is soft, or the gravel loose, there must be
platform of timber laid under the masonry, to consist
two thicknesses of three-inch planks laid across ea
other, and if necessary have a row of pile-planki
driven all round the outside." The practice here d
scribed was constantly followed, the rows of sheeti
piles being always adopted if there was a current in t
stream where the bridge was built ; and if in consequen
the slightest danger was apprehended of the botto
being wasted or carried away. Whatever might be t
nature of the artificial bottom, this piling was nev
neglected where circumstances seemed to require it ; an
as these will rarely be found precisely alike in any tw
situations, the expediency of piling must be regulat

according to the necessity of each particular case. In his general specification Mr. Telford goes on to say, " If the ground is hard, instead of a platform under the foundations there must be an inverted arch or pavement laid across, and wedged between the abutments, as wide as the bridge, well secured above and below by rows of stones sunk deep in the bed of the river." (This method of securing the inverted arches or pitching is shown in figs. 9 and 10.)

Such were his general directions with regard to foundations, the power of making alterations according to circumstances being reserved as before described. Connected with the subject of the Highland roads and bridges, Mr. Telford has left a most valuable scale of proportions, which he adopted in those works, and which will be found to combine elegance and beauty of design with strength and safety of construction. The proportions are given for all arches from four to fifty feet span, and as it will prove a most valuable guide to engineers, road surveyors, county surveyors, and all others who are concerned either in the repair or construction of bridges, we make no apology for inserting the Table in this place.

Span of arches.	Rise of arch from the springing.	Depth of arch stones.	Height of abutment from the bed to the springing.	Thickness of the abutment walls on an average.	Length of parapets from the face of the abutments with wings under.	Height of parapets above the crown of the arch.	Thickness of spandrils and wings on an average.	Thickness of inverted arches, where necessary.
4	1·6	1	2·6	1·6	9	1·2	1·6	0·9
6	2	1	2·6	2	10	2·2	1·6	1
8	3	1·2	2·6	2	12	3·2	2	1
10	3·6	1·3	3	2·6	12	3·2	2	1
12	4	1·4	3	3	14	3·2	2·6	1
18	6	1·6	3	4·6	18	3·2	2·9	1·4
24	8	1·9	4	5	24	4·2	2·9	1·4
30	12	2·0	4	5·6	30	4·2	3·0	1·6
50	15	2·6	6	6	36	4·8	3·6	1·6

It ought to be observed that this Table furnishes a system of proportions which are perfectly safe, without requiring an extravagant expenditure. Probably Mr. Telford, in framing his rules for the strength of bridges, deserves more credit than any engineer who preceded him, because he always acted on the principle of constructing substantial works with a due regard to cost. It will be found, after an attentive examination of all the works executed by this great man, that the two material elements of *necessity* and *expense* were ever most judiciously balanced one against the other. Whilst on the one hand we see no extravagant outlay incommensurate with the necessity of the case, on the other hand we can point out no necessity unprovided for.

No. 3.

In considering foundations of sand it is necessary to observe that great differences will be found between the various stratifications comprised under the name of "sandy." The sand which forms the prevailing ingredient of such soils may be either rounded or angular, the latter forming what is technically called a sharp sand. Again, the particles of sand are sometimes exceedingly minute, as in very fine sands; or very large, as when the sand approaches the nature of gravel. Sandy soils are also distinguished from each other by the proportion they contain of clay or other earthy matter, which, in combination with the sand, is termed silt. Fine sharp sand is usually found in currents sufficiently rapid to carry away the foreign particles

commonly deposited with the sand in still waters. The beds of streams with currents still more rapid may consist of gravel, and the sand found with this will be of a sharp angular nature. In proportion as the current is strong, so will increase the coarseness of the gravel, and so will diminish the quantity of sand mixed with it; but in the beds of such streams the sand will always be sharp, and very far removed from the nature of silt. Much has been said and written on the subject of quicksands, and it is usually considered that great danger attends the experiment of founding upon them. A quicksand is formed simply by the action of water on a bed of this material, whether of a silty or pure sandy nature; and the great danger of its giving way when any weight is placed upon a quicksand arises from its tendency to escape with the water, and pass from under the pressure. But as this kind of soil is, when in a state of rest, remarkably solid, all the interstices being filled by more minute particles of sand, it affords a very excellent foundation when it is confined in such a situation that it cannot possibly escape. Thus a quicksand surrounded by a strong and close encaissement of piling would be perfectly safe as a foundation. Probably the most compact and solid description of sand is that called silt, because even the smallest vacuities are occupied; but at the same time it is found, by reason of its small specific gravity, to be the most easily disturbed and set in motion by water, and therefore the most difficult to deal with. Specimens of such quicksands are met with in the beds of most rivers where the current is of mo-

derate velocity, as in some of the reaches in the Thames, the Crouch, the Blackwater, and other rivers on the eastern coast.

Sands of a sharper kind also are not unfrequently found in the form of quicksands, but when undisturbed by the motion of water are sufficiently solid to bear the weight of any structure. Experiments on the peculiar nature of quicksands may be made on any fine sandy beach, or on any tract of fine sand which is dry at low water. It will be found that almost as soon as the water sinks below its surface, carriages may be drawn over in safety almost without leaving an impression. But let any weight, having irregular surfaces, be placed on this kind of sand, and the projecting parts will sink into it; then cause this weight to be slightly moved by hand, or in any other manner, the sand will immediately give way, and, if mixed with water, be set in motion, and a real quicksand will be thus produced. It is not often found that clean dry sand, however coarse it may be in quality, will sink with the weight of a building; it may, however, be forced laterally out of its position, when the pressure is very considerable, and thus cause a sinking of the structure: therefore, when any danger of such a result is apprehended, the first part of the foundation should consist of a timber platform resting upon the sand.

The platform may be laid with sleepers ten inches wide and six inches thick, placed at the distance of three feet apart from centre to centre; these must be closely covered with four-inch thick planks, close jointed and secured to the sleepers with trenails. This platform must be of

sufficient superficial area to extend two or even three feet on each side beyond the base of the structure raised upon it. The exact superficial area, however, of the extension must be regulated by the height and solidity of the building, and the possibility of lateral space existing into which the sand foundation might be forced; but in any case where a building has to be erected on a dry sand foundation, it may be made perfectly safe and unyielding to any weight that can be placed upon it by adopting the platform covering, as described, and of sufficient superficial area, the dimensions of which area must be determined according to the circumstances above stated.

In England, where the sand, whether coarse or fine, is usually composed of hard silicious particles, very little danger of settlement need be feared; but where, as in the bed of the Seine, the sand or small gravel consists chiefly of rounded pieces of chalk without much admixture with any binding material, it is extremely probable that settlements would occur, as the effect of great pressure would be to crush and break the softer particles, and thus cause the under stratum to occupy less space than before the interstices were filled.

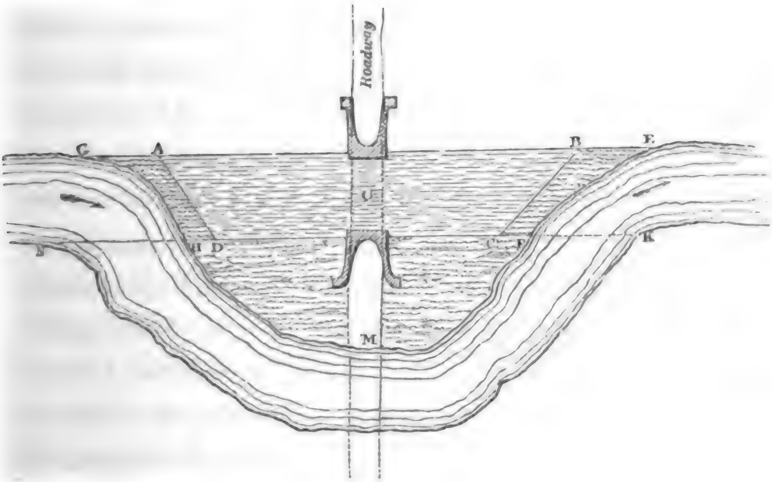
It may be concluded that almost every description of sand might be safely built upon if it could be protected from the action of water, which, when set in motion, is quite capable of carrying away such material, and of pressing it from under any superstructure which may rest upon it. In the peculiar case of bridge foundations, however, it is very seldom that a sand stratum so safely

circumstanced can be met with, because, being usually in the neighbourhood of large bodies of water, and the sand being of a porous nature, water will always be found filtering through, so as to render it an unsafe bed to build upon. Hence it is scarcely possible to find a bridge foundation established upon a sand bottom without the accompaniment of a platform, generally laid on piles. Notwithstanding all this, there are situations in which the practice of dispensing with timber on a sand bottom would be perfectly safe. For instance, where there is very little current in the river and the sand bottom has a covering of from two to three feet of clay or heavy gravel, which is never liable to be disturbed by floods or other causes, a building might safely rest on a close sand bottom. In other cases, an artificial covering may be placed over the sand, and thus prevent its disturbance. Such a covering also will often be found necessary after the erection of a bridge, in order to prevent a sandy or silty bottom from being carried away by the current; although before the erection of the bridge the current might not have had sufficient power to set the sand in motion. An instance of this kind occurred in the case of the Lary Bridge, at Plymouth, where the current being increased in velocity by the contraction of the waterway was found to act on the bottom in such a manner as to endanger the foundations; and in order to prevent serious consequences, Mr. Rennie adopted a covering of clay in the bottom, overlaid with stones varying in weight from two hundred pounds downwards, the whole thickness not exceeding two feet. This cover-

ing, which extended a distance of sixty or seventy feet above and below the bridge, became very solid, and effectually prevented the further wasting of the bottom.

When, instead of building a bridge in the existing channel of the river, it is considered eligible to build on the dry land and direct its course through a new channel,

Fig. 11.



as shown in the accompanying figure, it would be adviseable to proceed somewhat in the following manner.

The first operation to be performed is that of excavating the new river-course marked in the figure GEFH, leaving, however, a solid portion at each end of sufficient strength to prevent the water breaking through, and to admit a temporary road for the traffic during the time of excavating the new river-bed and abutment foundations, as represented in the figure by the letters GADH at the upper end, and the letters BEFC at the lower end, of

the stream. The earth taken out of the new channel may be laid in the seat of the new approach, or, if more convenient, on the small insular piece of land between the old and new channels marked HMF, from which place it may be removed into the old river-channel when the water is turned the other way, or it may be employed in completing the new approach.

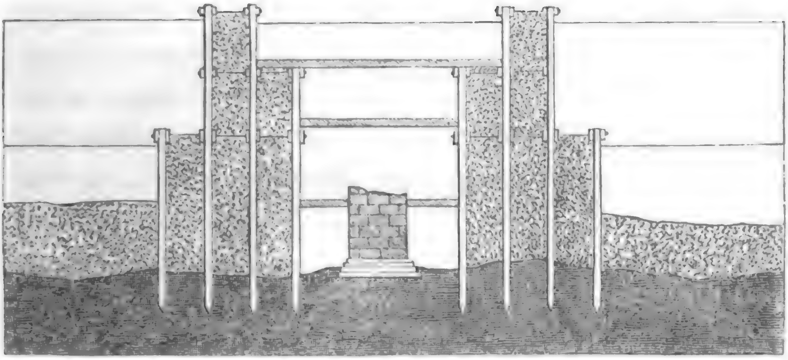
The engineer will decide upon the disposal of the excavated material according to the facilities of obtaining earth within a reasonable distance for making up the approach, in case that which is taken from the new channel should be used for filling up the old river-bed. As soon as the bridge has been built on the new cut in the situation marked O, the stank, or portion left at the lower end of the new cut, must be taken out, and afterwards the portion of earth marked GADH at the upper end must also be removed, and run by barrows into the old river-channel in the direction NH, in order to turn the water into the new course. Should the situation where this work is required be in retentive ground, which will not admit of water passing through it, the operation of turning the river may be very simple and attended with little difficulty. If, however, the stratification is of an open porous nature, as soon as the excavation is carried down as low as the water in the old channel, springs will be found to rise in the new excavation, and unless it can be drained by bringing up a level from the lower end of the cut at F, no better method remains of getting out the bottom than that of dredging with bag and spoon, or with a steam dredging engine.

It is necessary to observe, with reference to the expedient of turning a river in order to build on dry land, that the whole operation will be found very troublesome and expensive when the soil is porous and incapable of retaining water. In addition to the cost of getting out the new excavations under water, it is by no means unlikely that the same expensive foundation of piling and timber platform will be as indispensable now as if the bridge had to be built over the old channel; and the newly opened ground may, by the action of the water, be rendered so porous as to permit water to enter the openings for the piers and abutments to an extent requiring, before the building can be properly carried on, the construction of coffer-dams almost as expensive and formidable as if they had been made at first in the middle of the river. Hence it is necessary to use considerable caution in deciding upon opening new ground to build the bridge upon. Pits ought, if possible, to be sunk; indeed, they are absolutely necessary in all cases of large bridges; they alone will afford the engineer the proper means of comparing the different soils he has to contend with, both in the new and the old channels, and his judgment should of course be formed only after a due consideration of these and other particulars which he will acquire in the course of his investigations. Borings are not to be depended upon, and where the stratification is at all doubtful they ought never to be attempted. Trial-pits should be sunk as deep as to satisfy every inquiry.

In pursuance of the intention formerly expressed, w
proceed to give some details for the construction of
coffer-dam. By way of example, suppose it has bee
determined to place a bridge pier in a tide river, whe
there is ten feet depth of water at the lowest spring tide
The bottom has been found to consist of twelve fe
loose gravel and sand, with clay underneath. In th
situation suppose the depth at high water to be not le
than twenty-eight feet, making the whole depth, fro
surface of high water through the loose bottom down
the clay, forty feet. It will be necessary, in order
form the coffer-dam under these circumstances, to dri
four rows of piles all round the space to be built upo
The clay puddle will then occupy three distinct space
called puddle walls, between the four rows of piles. Th
lengths of the piles may be as follows:—outer row to b
driven down to within a foot of low water mark, and fi
feet into the clay, making their length twenty-eight fee
the two middle rows to be also driven five feet into th
bottom, and to stand three feet above high water mar
making their length forty-eight feet; the inner row
piles to be driven to about eleven feet above low wat
mark, and five feet into the clay, so that their leng
will be thirty-eight feet. The plan and section of th
dam, fig. 12, will show the form of construction, an
the following short outline of a specification will rend
the whole intelligible. The clear breadth between eac
two rows of piles to be six feet; the outer row of da
piles to be half logs of twelve inches by six inch

TRANSVERSE SECTION, FROM A TO B.

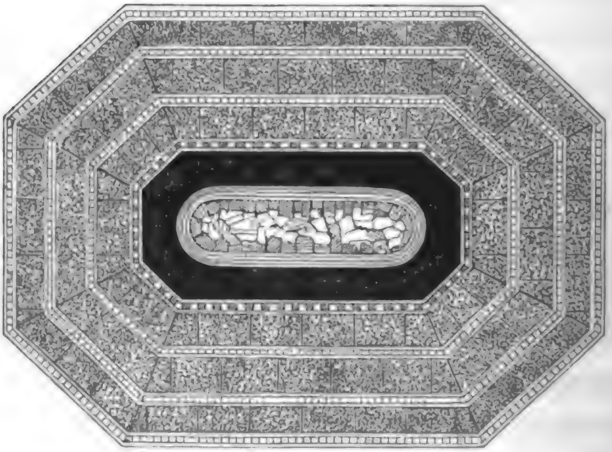
FIG. 12.



scantling; the middle rows of piles to be twelve-inch square logs; and the inner row of piles to be twelve by eight inches scantling. The whole of the timber for the piles to be straight grown, and to consist of the best Memel logs. A double row of waling-pieces to be placed all round the top of the inner piles, as shown in fig. 12, and to be connected by wrought iron bolts an inch and a quarter square. The bolts must have good heads, one inch thick and three inches square, and the other end must have a fine screw-thread cut upon it, with a three-inch square nut, one inch thick, made to screw on at the end of each bolt. Also between the timber and each of the nuts there must be a plate or disk of iron, called a washer, to prevent the nuts from pressing into the timber when tightly screwed up, and also to prevent the friction between the timber and the nut, which would be very considerable if the latter were screwed in contact with the wood. In the same manner several other double lines of waling must be fixed round

PLAN OF COFFER-DAM.

FIG. 13.



the whole coffer-dam, precisely as shown in the tran verse section, fig. 12. The whole of the waling to b of the same quality as the timber for the piles, and to b of scantling equal to one foot by six inches.

Connecting bolts must be introduced at intervals four feet from bolt to bolt all round the dam, as show in the figures.

In the situation we have supposed for the constructio of this dam, where a stratum twelve feet in depth sand or gravel rests upon the clay, it will be necessary excavate the whole of the former, which, on account its porous nature, would otherwise permit the water penetrate through it into the dam. This will occu however perfect and water-tight the clay filling place above the porous stratum may be, because the water wi pass under the water-tight material, and fill the interi

up to the level of the water outside. Hence the necessity not only for driving the piles through such a stratum as this, but even of entirely removing it, in order that the artificial filling-in between the piles may commence by a perfect junction with the clay beneath, and be carried up tight to the necessary height. The whole of the porous material therefore must be removed from the space to be occupied by the dam; and it will be found most convenient to effect this before driving the piles, because there will then be no obstruction to the working of a dredging engine. With respect to the filling-in between the piles, it is very important to distinguish with accuracy the material which is proper for this purpose from other kinds of earth, which, although known by the same general name, clay, would yet be found quite inefficient for the puddle of a coffer-dam. In specifications for these works, the usual expression is that good retentive clay must be filled in between the rows of piles, and on the judicious performance of this very indefinite injunction depends the tightness of the dam. Considering only the two extremes of very hard and very soft plastic clay, it will be found that the former of these, when broken up and thrown in between the piles, will seldom or never form a perfect dam. On the contrary, vacuities will remain between the broken pieces, and it will be found exceedingly difficult to beat down clay of this kind into a body sufficiently firm, compact, and solid to resist the efforts of the water to penetrate through it. If, again, clay of a very soft plastic nature be introduced, it will partially dissolve and com-

bine with the water when thrown into it, so that the space between the piles will be filled with a kind of mud puddle almost in a fluid state, of no greater consistency and no greater capability of keeping out water than mud itself. It is evident therefore that either kind of clay by itself would not answer the purpose intended of forming a solid water-tight puddle. All the clays, when used in a coffer-dam, require a mixture of gravel and sand, or a portion of pounded chalk will be found an excellent material to give solidity to the soft portion of the clay, and to fill the vacuities and interstices which may be expected to exist where the clay is of a hard and lumpy description. However general may be the opinion, it is certain that one more erroneous was never entertained than that clay alone is a proper material to make a good puddle-dam. Clay by itself is subject to great changes, according to the alternations of heat and cold, drought and moisture. In very dry weather, and when exposed for a time to the influence of the sun, all moisture will be extracted; and the clay will invariably crack and separate into a number of irregular fragments, which will never afterwards unite so as to form an adhesive water-tight substance. The difficulty of compressing clay, when placed in a dam of any considerable depth, into a solid mass without hollows has been already noticed. If in addition to this objection we consider the immense weight and pressure of clay so compressed against the piles forming the sides of the dam, and the consequent strain on the piles, which ought only to be employed in resisting the pressure

of the water from without, we shall see sufficient reason to decide, on these as well as on other grounds, against the practice of puddling entirely with clay. From the very best information which can be brought to bear on this subject, namely, that derived from long and watchful experience, accompanied by the knowledge that he has himself, as a contractor, lost large sums of money on account of too great a faith in clay puddles, the writer is enabled to speak very positively on the nature of this material, and in addition to the objections already advanced begs to add his own personal observations of the fact that puddles composed entirely of clay have usually bulged, given way, and been found incapable of keeping out the water when of considerable depth, and that in any case a puddle with an admixture of gravel, chalk, and sand will make a safer water-tight dam than clay alone. In order to secure a proper construction of coffer-dam, we recommend that all specifications should very fully particularize the manner of filling the coffer spaces; thus, the kind of clay to be used,—the quantities in which it is to be thrown into the dam,—the size of the pieces into which, if hard clay, it must be chopped,—and the mixture which is to be added, whether of sand, gravel, loam, mould, or chalk,—should all form particular instructions in the specification. It is certain that in great depths of water the practice ought never to be dispensed with of mixing some foreign material with the clay, otherwise it will not set or become hard under water. We have mentioned sand, gravel, and chalk, as proper materials for this purpose, but it is impos-

sible to assign any proportions to be adopted in the mixing, because these can only be determined as a matter of judgment on seeing the kind of clay to be used, knowing its tenacity and capability of resisting pressure, and also whether in the state it is dug it contains already any mixture of gravel or other foreign ingredient. Probably three parts of pure clay, two of chalk, and one of fine gravel, would make an excellent compound for filling the dam. These materials, however, should be well mixed together, the chalk and clay chopped small, and no stone larger than a hen's egg allowed to pass amongst the gravel.

It is usual in large dams to cover the top of the puddle with bricks, but a foot in depth of good strong gravel, grouted with lime, is much cheaper, and answers equally well with the covering of bricks.

All the piles used for the dam must be shod with good sound wrought iron shoes, weighing not less than 10 lbs. each, and hooped also with iron of the same kind by rings three inches broad and three quarters of an inch thick, to prevent the timber from splintering or otherwise giving way under the driving.

Having dwelt thus at large on the form of construction, and described in detail the component parts necessary for a first-rate coffer-dam, it might be thought by some that the following brief remarks on the means of carrying into effect the actual working operations of forming a coffer-dam are too exclusively within the contractor's province to be of any use to the engineer. But, on the other hand, considering how great is the difficulty

of drawing a line to separate the duty and responsibility of the engineer who directs from that of the contractor who executes, and, further, admitting that the superintendent of a work cannot be too intimately acquainted with the details of execution even independently of the materials to be dealt with, some such information, with reference to coffer-dams, can hardly be thought uninteresting to the engineer.

The variation of circumstances under which coffer-dams are constructed will occasion differences in the facilities, and generally in the entire mode, of execution, because the same means which in one case may be successfully adopted for driving the piles will be quite inapplicable in other situations where the difficulties are perhaps much greater. For example, in many rivers and canals it might be possible, without obstructing the navigation, to erect a temporary fixed stage, on which the pile engine could stand, and proceed without interruption in the work of driving the piles. But in a tide river it will scarcely be possible to drive from a fixed stage, and it is therefore necessary to employ barges or other floating vessels on which the pile engines must be erected. The barges used for this purpose are generally of forty or fifty tons burden, and by proper management of these the pile engines may be made to work at all times of tide.

The first operation in constructing a coffer-dam should be that of driving the guide piles, which are so called, because, being the first which are driven in each row, they serve as a guage and a guide both in position and

scantling for the rest of the piles. These guide or guage piles are usually placed about ten feet apart, and as they are driven down without any marks to indicate their proper position, except the known direction of the sides and ends of the dam, and of course are driven before any waling can be fixed to preserve them in a perpendicular direction, the business of driving them is one of more difficulty and complication than that of inserting the intermediate piles.

In order to drive these guide piles, the vessel containing the pile engine should be moored stem and stern alongside the line of the intended dam, so that as many as possible of the piles can be driven without altering the position of the vessel. In this first part of the process a ringing engine and wooden monkey of about eight hundred pounds weight will act with more success than a heavy iron ram, and when it is found that the piles will not go deeper without skewing from the perpendicular direction it is better to discontinue driving them, and leave them to be forced down afterwards when the others have been driven. As soon as all the guage piles of any one row have been fixed, the walings should be fastened to them, and the intermediate piles, ten in number between each pair of guage piles, may be driven down. It is scarcely necessary to say that the waling will be of important service in driving these piles, because it will determine their precise position, and serve to keep them upright during the process. Considering the great inconvenience and delay in the execution of a work which may arise from an imperfectly constructed coffer-dam,

we would recommend that the engineer should always direct the contractor in his specifications to place all the piles intended to be used in some convenient spot for inspection before they can be admitted into the dam. The piles may then be laid close to each other, their scantling, and particularly their breadths, properly measured, and their angles tried in order to see that they will fit close to each other, and fully occupy the space between the walings. This preliminary fixing is just as necessary as that adopted in trying roofs and centres before they are actually erected, and if insisted on by engineers, in the case of coffer-dams, would prevent many evil consequences which may be traced entirely to the use of improper piling in the dams. It is certain that if piles laid down on the ground side by side before being driven will not fit close to each other, they never will afterwards, and under such circumstances it will ever be found exceedingly dangerous to try them.

When the engineer is satisfied that the piles are properly prepared, and can safely be used in the work, the whole number between two of the guage piles should at once be placed upright for driving, instead of being inserted and driven down singly. This method will in a great measure prevent the piles from taking a sloping direction, and will ensure the desired result of properly filling up the spaces between the guage piles with a close-fitting compact row of intermediate piles. We should recommend that the driving be commenced and continued as long as practicable with a well-constructed ringing engine, and that when the piles cease to sink

fairly after repeated blows from the monkey, a heavy iron ram, with considerable fall and weighing fifteen or eighteen hundred pounds, be substituted to complete the driving. Care should be taken not to allow the heads of the piles to get bushy or besomy; and when this appearance is observed, the heads should be immediately squared off, in order that the force of the blow may not be deadened, but may produce its full effect by falling on a solid substance. If this precaution be neglected, the driving cannot be done without great difficulty.

Before deciding on the particular kind of foundation which should be adopted under any given circumstances, the engineer ought to inform himself with every possible care as to the nature of the bottom, not only as he may find it at the time of his examination, but also with reference to the possibility of future changes, which might entirely alter its character.

The necessity for such an extended consideration will be evident from an observation of the effects produced by water on all stratifications with which this powerful agent is found in contact.

We have seen that sandy soils, when saturated with water, become quicksands; and if we extend our notice on this subject to other soils, we shall find changes of structure no less remarkable in clay, chalk, peat, &c., according as these are saturated in a greater or less degree. The difference between hard dry compact clays and the softer kinds, which may be dug with an underhand grafting tool, is sufficiently remarkable. With respect to chalk, we may distinguish that upon which

water has extensively operated since its formation as the substance known by the name of chalk-marl, a material widely different from the dry beds of hard flinty chalk which more commonly prevail. The effect of water upon peat also may be considered identical with the production of ooze, which is nothing more than peat in which the vegetable structure is entirely destroyed and the peat converted into a soft and very compressible substance, sometimes of a consistency not much harder than butter. Now, as it is a fact of very common observation that great changes take place in the circumstances under which water is found pervading various strata, it becomes obviously of importance to investigate thoroughly all these circumstances before deciding upon the method of constructing an important foundation. This remark demands particular attention whenever it is in contemplation to introduce a timber foundation, because, independent of the settlement or otherwise giving-way of the stratification, which the changes we have alluded to may occasion, it is well known that the change from a state of moisture to that of drought produces the most injurious effects upon all timber. Thus it is found that timber foundations originally laid down in wet marshy districts, which have afterwards been drained, have decayed and endangered the stability of the superstructure; the same result has followed in many situations also where the water has receded from natural causes, or where wet and dry seasons affect the moisture of the ground. In the neighbourhood of large towns the sinking of

numerous wells will frequently withdraw water from a timber foundation and thus cause its decay, while, if the original state of moisture had not been disturbed, the foundation would have long remained perfectly sound. Even the temporary variations from wet to dry, occasioned by the ebb and flow of the tide, are found very injurious to piling. The decay of timber foundations in the situations we have been describing is made apparent in various ways, but usually by cracks or fissures from top to bottom of the building, occasioned by the want of support in those places where the timber has decayed. These cracks will be more or less extensive according to the height and weight of the building, but in any case are dangerous, to say nothing of the disfigurement they produce. In order to guard against this decay of the timber, the foundation ought, if possible, to be laid low enough to ensure the object of keeping the wood-work always moist, wherever any danger exists of the water being withdrawn. To effect this, however, would often occasion a vast expense, and it will then be found most expedient to abandon the idea of using timber in the foundations.

Before quitting the subject of piling, we feel it necessary to advert to a most injudicious practice which we have seen adopted in many situations where the bottom is considered unsound. We allude to the system of partial piling, which is frequently used in parts of a foundation, while in other places of the same foundation the ground is considered sufficiently hard to bear the necessary weight without the security of piles. The

obvious effect of this system is to produce in the foundation certain very hard points of support, while the remaining surface of the bottom, namely, where no piles have been driven, is usually compressible, and unable to bear the great weight upon it without yielding in some slight degree. It must be observed that settlement or sinking will occur in almost all buildings, but provided this be perfectly uniform in all parts of the foundation the consequence to the building is by no means injurious. But conceive one part of a foundation yielding even to an extent ever so trifling, while other contiguous parts remain perfectly solid and immoveable, and we may readily arrive at one very common cause of settlement in buildings. The part of the foundation which sinks under the pressure of the building may be considered as a hollow lying under the stone-work, while the effect produced upon each course of stones may be compared to the strain upon a beam projecting from a wall and loaded uniformly with a weight. Each course of stones is in fact a beam, supported at one part on the hard piled foundation, while the weight of the building above, tending to press the other part down into the yielding part of the foundation, acts with great leverage, and usually produces a fracture.

To investigate the direct tendency to fracture occasioned by a partially unsound bottom, and thus show in the clearest manner the danger of allowing so great a weight as that which usually presses on the piers of a bridge to act in such a destructive manner, would by no means be difficult; but as this would be more in the

nature of a theoretical inquiry than the object of th
paper will justify, it may be sufficient to point out t
manner in which this kind of destruction is caused.

At present the practice of partial piling is notorious
common, particularly in the neighbourhood of the metr
polis and other large towns, where the builders n
unfrequently take great praise to themselves for t
judgment they exercise in order to determine what pa
of the foundation for walls, &c., should be piled, a
what parts may safely be built upon without pil
Such an injudicious system cannot be too strong
condemned, and engineers and architects ought
be very careful how they trust to the management
contracting builders, and others having charge und
them of extensive buildings. In order to show the co
sequences of even a very slight partial settlement in
building, as illustrating the danger of adopting a syst
which is actually calculated to produce such a sett
ment, we shall relate an instance with which the wri
himself was intimately acquainted. The piers of a la
aqueduct, eleven in number, with two abutments, h
all been founded on gravel, a few feet below the surfa
and stood remarkably well, the masonry appearing wi
out a flaw when they were carried up to their full heig
of about fifty feet. One of the piers at the south e
however, was founded one part on the gravel and t
other on very hard Whinstone rock, the surface of whi
was merely levelled and the building at once commenc
When carried up to about thirty feet a formidable fiss
was observed from top to bottom of this pier, and t

only possible source to which the mischief could be traced was the step of founding the pier partly on the rock and partly on the gravel. Had the whole pier been on the rock, it would of course have stood without any settlement; had the whole been on the gravel, it would perhaps have settled to a trifling extent, but would no doubt have stood as well as all the other piers, which were founded entirely on the gravel. Placed, however, partly on the rock, which was perfectly solid, and partly on the gravel, which slightly yielded beneath the great pressure upon it, the consequence followed as described above; in order to remedy which, a great number of the stones had to be taken out and re-set on each side of the fissure. The application of this example, which shows the consequences of building on a natural foundation more solid in one part than another, to the case of an artificial foundation put in precisely the same situation by the injudicious use of piling, is sufficiently obvious. In defence of the practice of partial piling it has been urged that the ground in its original state is already more solid in one place than another, and that the piling is only introduced to render the whole surface equally firm. But the answer to this obviously is, that the introduction of one evil is no cure for another, and it would probably be found better to build on the original ground, bad as it is, than to adopt a foundation so unequally sound as that which is formed by the use of piling in occasional spots. Where the unsound parts of the foundation are of small size, a few flat-bedded stones placed in the bottom will frequently answer as well as

piling; but if unsound to any considerable extent, the engineer will prefer the use either of piling or of concrete all over the foundations to the imperfect and unsafe expedient of patching up the unsound parts with piles. Looking at the usual effects of partial piling in a suspected foundation, it would seem to be really worse than useless, and the obvious rule pointed out by experience appears to be that piling, whenever adopted, should extend entirely over the whole base of the foundation, and even some few feet beyond it all round. The piles should be driven at equal spaces apart, and regularly planked over, as described in a former paper.

Some French engineers practise a system of rendering the ground to be built upon uniformly solid by means of pounding with a heavy rammer, and wherever, by this process, any part is beaten down below the general level, the space is filled up with other material, until the whole is well compressed and brought to a level surface. With some kinds of clay this system of pounding may answer very well, but in other stratifications it would be rather injurious than otherwise. For instance, where intermixed beds of clay and sand form the stratification it is frequently found that some parts are less solid than others, and yet the practice of pounding would probably produce no good effect upon the defective parts, as it would be very apt to occasion water to spring up from the porous strata beneath. Thus in building on almost any part of the London clay, which contains numerous beds of sand, it would by no means be judicious to try the expedient of pounding. We are inclined to think

that dry clays, mould, and newly made ground, present the only cases where pounding would be of any advantage in effecting the object of rendering the ground uniformly solid.

No. 4.

The value of concrete, as a substitute for stone or timber in foundations, is now well known, and the situations will be found exceedingly rare where this substance cannot be successfully employed. If tried, however, on a quicksand where the water could carry away the particles of sand, or on gravel where the finer particles could be similarly acted on by water, and the whole stratum thus lowered, hollows would be formed under the concrete which would in all probability give way and much endanger the stability of any building resting upon it. In a situation of this kind piling would be infinitely preferable to concrete, and in like manner it would be better to use piles than to lay concrete on any soft stratum into which the concrete would sink. The cheapness of concrete is one great recommendation which ought never to be overlooked. When an unsound bottom has been tried, and the fact ascertained that its depth is inconsiderable, say five or six feet deep, it would be on the one hand very bad engineering to attempt building with stone upon the unsound stratum itself, and on the other hand, on account of the expense, the error in judgment would scarcely be more justifiable of taking out the unsound stratum and building with stone from the solid; but this is a case where concrete may be used with great advan-

tage, because it may without danger be placed on
unsound stratum; and in a much greater degree
concrete foundations be superior to all others where
depth of the unsound strata is considerable, and wh
except for the concrete, the extensive use of piling wo
be absolutely required.

Objections have been urged against the use of conc
in marshy grounds, and supported by allusion to
Greenwich Railway viaduct, the piers of which w
founded on concrete, and some considerable settlem
afterwards took place. The writer is not sufficie
acquainted with the details of that work to be able sa
factorily to explain the causes of the settlement, bu
certainly not inclined to attribute any evil conseque
to the use of the concrete in that situation, because
is persuaded that this material is the best that c
have been adopted for a foundation in those mars
This opinion is corroborated by the fact that the C
don Railway viaduct, which joins the Greenwich Rail
about a mile and a half from London Bridge, an
founded in the same marshes, even in the very w
part of them, stands remarkably well, and presents
trace of any flaw, crack, or settlement.

Numerous examples might be pointed out w
buildings of great extent are found to stand remark
well on concrete foundations, placed, too, in a stra
cation far from solid. Amongst these may be m
tioned the Penitentiary at Mill Bank, and the very l
building called Fishmongers' Hall, on the north
of London Bridge. Both these buildings rest o

substratum of very soft elastic alluvial soil; and yet their foundations, of concrète, are unrivalled for firmness and stability. The Penitentiary, particularly, is an old building, and may be cited as one which has been fairly tried by the test of time. Pieces of concrete which have occasionally been taken from the foundation of this building are not inferior in hardness to some of the pudding-stone rocks found in the neighbourhood of the coal fields.

The kind of limestone from which the concrete is made very importantly affects its quality. The following are the principal varieties of strata from which limes are burnt in this country.

1. The upper and lower chalk, extending over a considerable portion of Lincolnshire and Yorkshire, ranging also through the counties of Norfolk, Suffolk, Hertford, Bucks, Wilts, Dorset, Hants, Surrey, Sussex, and Kent. 2. The Ashburnham limestone, deriving its name from the locality where found in the county of Kent. 3. The oolite formation, in which are numerous beds of limestone, throughout the counties of Dorset, Somerset, Gloucester, Oxford, Northampton, Rutland, Lincoln, and York. 4. The lias formation, extending from Lyme Regis, in Dorsetshire, through the counties of Dorset, Somerset, Gloucester, Warwick, Leicester, Nottingham, Lincoln, and York. 5. The magnesian limestone formation, extending over the eastern parts of the county of Durham. 6. The carboniferous or mountain limestone, extensively prevailing in the north of England, in Cumberland, Northumberland, Yorkshire, Derby-

shire, also in Denbighshire, Monmouthshire, Glamorganshire, and Somersetshire; and, lastly, the Silurian rocks, in which are numerous beds of limestone, particularly at Wenlock, Dudley, Llandovery in Carmarthenshire, &c. It is true that limestones are found in other formations besides the preceding, for instance, occasionally in the old red sandstone, and more rarely beds of limestone occur in the coal formation. We have, however, enumerated all the rocks from which limestones are usually quarried for the purpose of being burnt into lime. The qualities of numerous limestones appear to have been well considered by Colonel Pasley, and of all those mentioned above he gives a decided preference to the lias limestones, when speaking of the best description of lime for concrete. This author condemns the use of the pure limes, which do not possess the property of setting readily under water, because, in such large masses as are requisite for the practical purposes of forming artificial foundations under heavy buildings, the concrete made of such lime would never have time to set in a damp situation, and wet would destroy it. The limes of Dorking, Merstham, Reigate, and Halling, however, which are all procured from the beds of the lower chalk formation, usually called the grey chalk, possess hydraulic properties in a greater or less degree, and will be found an excellent material for concrete. All of these places supply the metropolis with lime, which is, no doubt, extensively used for concrete. In the application, however, of lime burnt from chalk, care should be taken never to confound the upper with

the lower chalk, because none of the beds containing flint possess the qualities of water limes. In fact, the lime from the upper chalk usually makes weak mortar, and therefore is quite unfit for concrete. Water limes are procured at several places in the range of the Sussex South Downs, as at Poynings, Clayton, and elsewhere in the neighbourhood of Brighton. It is usually considered that the magnesian limestones possess hydraulic properties, but, as an ingredient in concrete, they are certainly inferior both to the lias and to the stronger of the grey chalk limes.

It seems that all the water limes of this country contain a certain portion of clay, and of this ingredient the analyses of some of the magnesian limestones show as much as eleven per cent. The carboniferous or mountain limestones, abounding so extensively in the neighbourhood of the coal and iron districts of Great Britain, burn into a very strong and excellent lime. It must, however, be observed, that limestones generally require time and fuel for burning in proportion to the strength of the lime they yield; and this, again, in limestones of the same composition, is usually in proportion to their hardness. The chalk seems to be an exception to this rule, which certainly applies to all other limestones with which we are acquainted. Probably the most easily burnt lime that can be found is made from the chalk marl of Norfolk, but this lime, although very pure in quality, is exceedingly weak. The ordinary dry chalk also burns with great ease, and the lime may be considered of medium quality; and, lastly, the grey chalk

burns with very little more difficulty, and the lime produced is excellent. Many valuable beds of limestone are met with in the oolitic series, particularly in the lower or great oolite, where those beds are frequently found resting on the white or yellowish-coloured freestone, from which, more particularly, the oolite formation derives its name. The oolite limestones most proper for burning into lime are mostly of a dark mottled grey colour, not unlike the lias; they burn with difficulty, but the lime is of excellent quality, and much valued for its strength and durability. The writer thinks it probable that the oolitic limestones in general possess hydraulic properties, but cannot speak positively on the subject.

The Silurian rocks furnish a great variety of limestones, some of which are so very impure, and so filled with large fossils, which have introduced animal matter into combination with the calcareous basis, as to render them quite unfit for being burnt into lime. In the neighbourhood of Dudley, however, and in other parts of the Silurian range, considerable quantities of lime are burnt. Considering the usually exposed situation of concrete foundations in regard to the action of water, and the consequent necessity that the lime used in its formation should possess to a considerable degree the properties of setting and remaining hard under water, it is always advisable, if possible, to use a strong lime, if not a regular water lime, as the lias. The mountain limestones burn into a lime which sets remarkably hard, and may safely be depended on in all concrete founda-

tions; on the contrary, none of the lime made from chalk marl, or dry chalk with flints, ought ever to be used in wet situations, since it will never set or become hard when continually subjected to water in contact with it. Before determining on the proportion of lime that should be used in making concrete, the quality and size of the gravel ought to be considered, because it must be observed that the quantity of lime should be increased according to the coarseness of the sand used with the gravel.

It is evident that in the composition of concrete the sand or the finer particles of gravel combine with the lime as in ordinary mortar, and it seems reasonable that the quantities of quicklime and sand should bear the same proportion to each other in good concrete as in good mortar, because the obvious condition sought to be fulfilled is that the pebbles of the larger gravel shall be cemented together in the most perfect manner. Again, as the fluid mass of water, lime, and sand must fill the entire of the interstices between the larger particles, the quantity of this mass must vary with the nature and size of the pebbles. It is not true that the larger pebbles placed in mass always contain more unoccupied space than the smaller pebbles, this being a law which only holds where the stones, great and small, are to a certain extent homologous and similar figures. It is clear, if the smaller pebbles are mostly rounded, and the larger mostly angular, the former may very readily contain more space or vacuity than the latter, the masses of each being equal. As a general rule, the gravel used for concrete should not be too fine, and, at the same

time, large stones are inadmissible. The bed of the river Thames in many places furnishes an excellent mixture of sand and gravel for making concrete, without any preparation whatever. Concrete used in the foundation of the Penitentiary was made of this Thames gravel, and certainly a better specimen of an artificial foundation can nowhere be met with. Colonel Pasley relates the fact that part of the foundation of the Penitentiary was found to be giving way; and the writer of this article is able, from an acquaintance with the history of this first use of concrete, to account for the giving way alluded to. Mr. John Hughes, the father of the writer, happened to be the contractor who executed the concrete foundation. In consequence of the interference of some of the inspectors or superintendents, Mr. Hughes was compelled, contrary to his own remonstrances and strong objections, to screen the gravel, and thus separate all the sand from it previous to mixing it with the lime to form the concrete. The consequence was one which might with great certainty have been anticipated; the lime mixed with the large gravel alone, without sand, even if it hardened at all, instead of setting into an artificial stone had no more strength than dried clay, and none of the solidity and strength of good concrete or artificial rock. Hence the reason of the failure, which was prevented from proceeding to a greater extent by a subsequent order to use the gravel as it was raised by the engine out of the bed of the river, without any preparation whatever. It follows from what has been said already of the composition of concrete, that the same quantity of pebbles in pro-

portion to the lime and sand must be used, whatever be the quality of these, and hence the subject which requires the most important exercise of judgment is that of determining the relative portions of lime and sand to be employed as the cementing ingredients, or matrix, with which to fill the interstices between the pebbles.

Thus the theory of composing the best concrete appears simply to involve the same consideration as that of making the strongest mortar with the same materials; that is, with the same kind of lime and sand employed for each purpose.

The pure kinds of lime from Dorking, Reigate, Merstham, Halling, Higham, Frindsbury, and other places in the neighbourhood, although not of such strength as the lime burnt from the lias, the carboniferous, and other hard limestones, will take more sand than the latter in combination with it in the state of mortar.

For instance, the chalk limes will usually take three times their bulk of moist sand, while the lias lime will not make good mortar if more than twice its bulk of sand be used; the quantity for the carboniferous limestones being about a mean between the two, or two and a half times its own bulk measured in a moist state, the lime being measured before slaking. This difference in the proportion of sand to make mortar with different limes appears to depend on the relative purity of the limes themselves.

Thus, the chalk, being the purest carbonate of lime with which we are acquainted, requires a greater quantity of sand than almost any other kind of lime; while

the lias, being an argillaceous limestone, would be overloaded and adulterated by a greater mixture of sand than here stated. It has been already noticed that when the sand used for mortar is of a coarse quality, a less quantity must be mixed with the lime than if the quality be finer; and this axiom will be found in perfect accordance with a law which would regulate, on very sound principles, the quantity of sand which should be used in making mortar, namely, that the sand should measure in a moist state as much, and no more, than when thoroughly incorporated with the quicklime and water. Should the sand for making mortar be too coarse, it will evidently not enter into that intimate combination with the lime which is necessary to produce the requisite adhesiveness and solidifying property of mortar. The cement or mixture containing too great a proportion of large sand may be considered as a putty uniting the coarser particles together, and if the quantity of fine sharp sand in combination with the lime is insufficient, the strength of the mass will not be greater than that of putty, or dried clay, as already stated. It would seem, however, that if fine sharp sand be used for the mortar in sufficient quantity, the addition of a certain proportion of larger particles can do no harm, except when the mortar is required for fine joints of brick-work or exterior masonry; in fact, when much rubble backing occurs in any building, it would be an economical plan to mix small pebbles with the mortar: by this plan a saving of lime could be effected, and the building would in every respect be as strong as if pure mortar alone had been

used. This exception, of course, does not in any way apply to concrete, where the mortar, consisting of the sand and quicklime incorporated with water, is used to cement the pebbles together. It is probable that grains of sand larger than a pin's head do not combine chemically with the lime, so that in all concrete a certain portion of the sand ought to be quite fine. Too much importance cannot be attached to the necessity of using sharp sand, free from earthy particles. The best possible kind of sand which can be used is composed of small grains of angular silex or quartz. The consequence of using sand adulterated with loamy and clayey particles is that the mortar will appear fat, as it is technically called by the workmen, while its real quality will be something intermediate between that of mud and good mortar. Several of the reaches in the river Thames furnish a sand which, although the grains composing it be smooth and rounded, is yet an excellent sand, and much esteemed by builders on account of its purely silicious nature, and its freedom from admixture with particles of clay, mould, or any other foreign ingredient. The London builder has also the advantage of being able to procure from the Thames sand of any degree of fineness or coarseness, according to the place from which the sand is taken; the principal circumstances which regulate the quality either of sand or gravel being the velocity of the current where it is found, and the nature of the streams falling into the river in the neighbourhood of the various beds of sand and gravel. To speak practically of the mixture of sand in making concrete, we

should say that none of the particles ought to exceed the size of a barley-corn, and that a greater portion of the sand should not exceed half that size, while another portion of the sand, equal in quantity to the other two portions united, should consist of grains of sand not so large as a pin's head;—then of such sand a quantity varying from two and a half to three and a half times the bulk of the quicklime may be used in making concrete. The lime should be measured in powder before slaking, and the sand should be measured in a moist state. With respect to the quantity of gravel or pebbles that should be used in the concrete, taking for our guide the law already laid down, that the lime and sand should exactly fill the interstices between the stones when laid as close as they can be without any admixture, it will be seen that such a quantity of lime and sand could be ascertained by experiment in the following manner: namely, take any cube measure of the pebbles from which the sand has been entirely separated, then, by pouring as much water into them as will entirely fill the interstices, the cubic measure of this quantity of water will give the proportion of lime and sand which should be used to any given bulk of the pebbles. It will be found, as the result of this experiment, that the quantity of water required to fill the interstices of the pebbles is something between one-third and one-half the cubical contents of the pebbles, according to the size and shape of the latter. Supposing the quantity to amount to one-half, and the sand, as previously determined, is to be equal to three times the quantity of lime, (which does

not increase the bulk of the concrete when properly incorporated with the sand,) the proportions of these ingredients,—lime, sand, and gravel,—will be as one, three, and six; that is, the concrete should consist of one part by measure of lime, three parts of sand, and six of gravel. Next, suppose the interstices of the gravel or pebbles to be equal to one-third their bulk, then the quantities of each ought to be—lime one, sand three, and gravel nine, parts. Mány concretes have been prepared according to each of these sets of proportions, but it will probably be found that the mean of the two, or the proportions of one, three, and seven, or seven and a half, will form a very excellent concrete with pebbles of the average size and lime of the average quality used about London. If, however, lime be used which only takes two portions of sand to one of lime to convert it into good mortar, the proportions for concrete will then be (where the vacuities are equal to one-half the cubical contents of the pebbles)—gravel four, sand two, and lime one; and when gravel is used in which the interstices have been found to measure one-third of the whole bulk, then the proportions will be six of gravel, two of sand, and one of lime.

Various methods of slaking lime are practised, and some engineers, particularly in Scotland, invariably require that the mortar shall be kept for some time after being mixed, before it is allowed to be used in the work. Even in concrete under heavy buildings it is desirable that the lime should be well slaked before being put into the foundation, because the bursting and expansion which

attends the slaking of the refractory portions of lime disturbs the whole mass of concrete, and prevents it from setting into a solid substance, as it ought to do. The practice of grinding the burnt lime into a fine powder before slaking is so very judicious that it should always be adopted: when thus ground, the lime should be equally covered over with the mixture of sand and gravel, in order to prevent, as much as possible, the escape of steam during the process of slaking; water may then be regularly poured on the heap from a watering-pot, and after allowing it to remain in this state about five or six hours, which will be sufficient time for slaking the ground lime, the whole must be carefully mixed together and thrown into the foundation as quickly as it can be got ready.

General specifications for concrete may run thus:—The gravel may be taken out of parts of the river Thames, or from other places hereafter described, but to be of such quality as the engineer shall approve of; the lime to be made from the lias formation, from the carboniferous limestone, from the under beds of the lime quarries of Merstham or Dorking, or from such other hydraulic limestone as the engineer may be satisfied with. If the Thames gravel cannot be conveniently procured, pit gravel, if properly washed, broken, and sorted, may be admitted, also broken flints, stones, and pebbles of any description, lime-siftings, broken bricks, or hard chalk, in the proportion of one-third of the latter to the whole bulk used. None of these materials to be larger than will pass through a ring of three inches diameter, and

they must be entirely cleansed from all clay, soil, or earthy matter of every description, but must be mixed with about one-third their whole bulk of pure sand, more or less, as the engineer may determine. The sand to be sharp and clean, and of the size and quality before described. The lime to be taken fresh from the kiln, and, after being ground in a mill, to be covered with the sand and gravel, and altogether prepared and used as already detailed.

Great difference of opinion prevails amongst architects and engineers as to the depth of concrete which should be adopted in the various situations where it has to be used as a foundation. To determine this it is of course necessary to be well acquainted with the nature of the strata on which the concrete is to rest, and even after knowing this it would be exceedingly injudicious to determine on any thickness which should be hazardous and doubtful. It may be safely stated, that the greatest depth will always be necessary where the foundation is of unequal solidity. Even the softest and most yielding substratum, such for instance as a bed of elastic peat, provided its compressibility be uniform over the whole base, will require a less thickness of concrete to support a building resting upon it than if the same building stood on concrete partly resting on peat and partly on a more solid stratum. As a general guide, the depth of concrete ought in no case of bridge or viaduct building to be less than three feet thick, and it will rarely be necessary to use a greater depth than six feet. Concrete being, however, much cheaper than any other kind of artificial

foundation,—whether of stone, brick, or wood,—th
can be no impropriety in putting any thickness that
necessary to raise the building from a solid base to nea
the surface of the ground, from which level a be
description of building becomes necessary.

In the course of the preceding papers, while c
sidering separately the various kinds of foundations,
have been led to remark on a few of the more comm
causes of settlement and failure in heavy buildings. T
we noticed that timber platforms, placed under a bu
ing, were subject to decay from the alternations
drought and moisture, from which arises the neces
of well observing whether the timber will be expo
permanently either to one or other of these states,
cause the changes from one to the other are exceedin
injurious to every kind of wood-work.

The agency of water in producing dangerous co
quences where a building is founded on sand has
been pointed out, and on this subject it will only
necessary to add our conviction of the great cau
which should be used by engineers in founding ei
upon an existing quicksand, or upon any sandy stra
which may by the future operations of water be c
verted into this state. We have further noticed,
point of great importance in all foundations, but p
ticularly those of stone, where unequal settlement is m
injurious than in concrete foundations, that it is m
essential to guard against the unequal solidity of
ground to be built upon, because it is in cases of

kind that unequal settlements of the building take place, and occasion fractures alike injurious to the appearance and to the stability of the structure.

The settlement in the wing wall of Gloucester Bridge affords an instructive example of the consequences which have followed an unfortunate inattention to this subject. One of the wing walls on the Gloucester side of this bridge is fractured from its base to the top of the parapet, where the opening is nearly three inches wide.

The cause of this settlement, as explained by Mr. Telford himself, appears to be that the abutment was founded thirty-three feet below the ground surface on a bed of strong coarse indurated gravel, while the wing walls joining the abutment, although they had to support the weight of a heavy embanked approach, were founded only ten feet below the surface on a bed of soft blue silt. In addition to this, the abutment had a timber platform placed upon a pavement of rubble stones in the bottom before the building was commenced, whereas the platform was entirely dispensed with in the foundation of the wing walls. The consequence of this imperfection is that the alluvial soil has given way, by reason of which the wing on one side, if not on both sides, at the Gloucester end of the bridge, has receded from the abutment, and a very large and unsightly fissure has taken place vertically in the line of junction. Whatever could have led to the unfortunate error which was here committed, of admitting so great an inequality in firmness of base for the wing wall as compared with that for the abutment, we are quite unable to conjecture, and the

circumstance is the more remarkable when we consi
the known ability and experience of the contractor v
was entrusted with the work. Mr. Telford, with
generosity worthy of his character, took the blame u
himself of being influenced by what he justly terms
injudicious parsimony of omitting a platform and pi
for the wing walls. It is probable, however, that
Telford himself, being at a distance from the place,
not so fully acquainted with the circumstances as
might otherwise have been, and as he afterwards ne
sarily became. Upon the whole, the case is worth reco
ing, in order that it may operate, in Mr. Telford's o
words, as "an useful caution to practical engineers."

We shall now relate an instance where an extens
building, consisting of several arches, was destroyed
a way which at first sight appears scarcely to be
counted for on ordinary grounds, but which, we thi
may fairly be attributed to the action of water.
case in question is of recent occurrence, and as
mention of the particular work might give rise to hos
feelings on the part of the engineers and others c
cerned, we refrain from making public the locality.
will be sufficient for the purpose we have here in v
to explain that the building was a viaduct, of considera
magnitude, placed across a valley, in the centre of wh
flowed a small stream. The upper stratum, on wh
the structure was raised, consisted of loose earth a
rubbish taken from the foundations of old buildin
The depth of this made ground was about five fe
under this was a bed of clay averaging about six feet

thickness, and under the clay a stratum of sand fourteen feet thick. The design for the building was well chosen, the proportions of all its parts were unexceptionable, and the workmanship of the brick-work and stone, laid partly in mortar and partly in cement, was of the very best description. The work had gone on progressively, and in a way quite satisfactory to all engaged, until nearly the last arch was thrown, when all at once two of the other arches fell, and a third was seen to be in so much danger that it had to be taken down in order to prevent a similar fate. All the piers and abutments had been founded on the bed of clay mentioned above, and we are strongly of opinion that the sinking of the piers and consequent falling in of the arches was occasioned by the action of the stream upon the bed of sand beneath the clay. The stream, it is true, flowed over the clay, which formed its bed, and was not, in the immediate neighbourhood of the viaduct, in contact with the sand; but we think it very clear, from the result which followed, that at some place not very far distant the water had a communication with the sand, by means of which the latter was set in motion, so as to leave an unsound space below the stratum of clay. A very strong proof that the mischief must be attributed to some change wrought below the clay may be derived from the fact that the clay on which the defective piers rested was only four feet six inches thick, while the general depth under the rest of the building was six feet. Besides this, under one of the piers which gave way the clay was dug out to the depth of one foot nine inches, and a bed of concrete

of the same thickness substituted in lieu of the clay, a under another of the piers, which also sunk, a layer concrete one foot in thickness was laid. Thus the wh thickness of clay between the bed of sand and the bott of the one pier was two feet nine inches, and between sand and the bottom of the other was three feet inches. These two piers gave way, while the other pie founded on the undisturbed bed of clay six feet in thic ness, remained firm and solid. It seems, therefore, t the sand, which extended under the whole building, be saturated by the water, which probably obtained acc during a time of flood, and which found an outlet escape when the flood subsided, was not able to supp the pressure of the clay when weighted by the buildi above, and it followed that the piers which were found at the least depth above the sand sunk down with clay, and caused the arches to fall as already describ On the other hand, the piers placed upon the full thic ness of six feet of clay stood without sinking; so t here we may suppose the mass of clay, although p tially hollowed by the defection of the sand, was sufficiently solid to support the weight of the build without material settlement.

Almost too obvious to need comment is the er which, as after-experience established, had been co mitted in the foundations of this viaduct. It will occ to the most unpractised student that had the clay b entirely taken out in the places where its depth was judg to be insufficient, instead of attempting to strengthen it concrete, and had the concrete been laid directly up

the sand and carried up to near the surface, where the building might have been commenced, we should not have had the present case of failure to cite as an example for future works. It might be worth while, however, in determining on the foundations of a viaduct consisting of a number of arches inconsiderable in span, to consider the cost of inverted arches between the piers as compared with that of concrete placed under them. The inverted arches should be eighteen inches thick, extending from pier to pier over the whole base of the structure, and the broad substantial bearing they afford will either entirely prevent settlement or will render it so uniform that no danger need be apprehended.

In confining our attention now to the general subject of settlements, independent of the particular nature of the foundation, we must not omit to notice the importance of firm solid backing for the walls of buildings which are required to withstand the pressure of earth. The wing walls and abutment walls of bridges form an important class of revetment or retaining walls of this kind, and, next to the actual preparation of the foundation on which they rest, the backing behind the walls claims the care and attention of the engineer. The object intended to be effected in the backing of walls is that of rendering the earth pressing upon the wall uniformly solid, in order that no irregular pressure may be allowed to act upon it. The most common form of imperfect and unsound backing is that in which water is allowed to lodge in hollows and crevices behind the wall, and thus to exert a pressure greatly in excess of that which the

wall is calculated to withstand, and greater than would ever occur in the case of dry earth behind the wall. From this we are led to observe the necessity of rendering the backing in a great degree water-tight, in order that it may act as a puddle-dam to prevent water from reaching the back of the wall. In considering the best material to be used for this kind of puddle backing, the same observations made in a former paper upon clay and other substances with reference to the puddle for coffer-dams will equally apply in the present instance. The cracks and fissures which attend the drying of clay, when much exposed to the sun and wind, are so exceedingly dangerous, as affording lodgement for water to press against the wall, that there is every reason to expect, at some time or other, fractures and dangerous settlements in walls which have been backed entirely with clay. It frequently happens that the wing walls of bridges which have stood for a long time, perhaps for many years, without showing any sign of fracture or decay, suddenly give way to pressure from behind, and in such cases the misfortune is usually attributable to the use of clay backing. Either the clay will have cracked so as to admit water throughout the crevices, or it will be found to have contracted in bulk and to have shrunk away from the wall, in each case leaving it exposed to the pressure of a column of water equal or nearly equal in height to that of the wall itself.

Although it is, therefore, very injudicious to adopt a puddle of pure clay, yet, if no other material can be procured, it will be found an excellent practice to dis-

continue the puddle at about eighteen inches below the top of the wall, and to complete the backing with a course of sand or fine gravel, which, resting on the clay, will serve to keep it moist and prevent it from cracking. It is also necessary in all heavy walls intended to withstand the pressure of earth, that small orifices, about four inches square, should be left at regular intervals in the face of the wall; these should pass entirely through the thickness of the wall, and be placed six or seven feet apart longitudinally, and about three feet apart vertically.

A grout puddle, which is very commonly used for the backing of lock and dock walls, is formed by a mixture of gravel, chalk, and a slight portion of clay or other adhesive material, with just sufficient water thoroughly to incorporate the whole mass. Puddles of this kind are far more to be depended on than clay puddles, since they are generally more water-tight, and are never known to crack.

In fixing the foundation of any building on a clay bottom, great care should be taken to place it sufficiently deep to be out of the reach of frost. This should particularly be attended to in the wing walls of canal and railway bridges, which in cuttings are usually made to step up the slopes. If the frost be allowed to act on the foundations of these bridges, an expansion and consequent fracture will take place, frequently to the extent of shattering and destroying the whole wall.

We had intended, in the present paper, to introduce some notices of the most important building stones and quarries from which stone is procured for the purposes

of bridge building in this country, but this has been rendered entirely unnecessary by the appearance of a very valuable Report from the Commissioners appointed by the Treasury to investigate the best kind of stone to be used for the New Houses of Parliament.

This report is in every way worthy of the talents and scientific attainments of the gentlemen who have prepared it, and probably a more useful and valuable body of information than that contained in the report itself, and the tables which accompany it, was never before placed within the reach of the engineer. Some idea of the labour and pains bestowed by the commissioners in the execution of their arduous yet interesting and instructive task may be derived from the fact that they have visited the localities, and made themselves perfectly acquainted with the nature and properties, of more than one hundred different kinds of building stone in England and Scotland. The information collected during their researches has been embodied in several tables, one of which describes each quarry which has been examined, and furnishes the following particulars:—The mineral designation of the stone; its component parts and colour—weight of a cubic foot of stone in its ordinary state—the entire depth of workable stone in the quarry; description of the beds and size of the blocks that can be procured.—Where known or reported to have been employed, and general remarks—prices of block-stone at the quarry; description and cost of carriage to the pool of London; cost of stone delivered in London per cubic foot; and cost of plain rubbed work as compared

with that upon Portland stone in London, per foot superficial, Portland being taken at 1·0.

Another table describes a vast number of buildings which the commissioners have also visited, in order to inform themselves as to the durability of the stone composing them.

The two last tables contain the results of experiments on the cohesive powers of a great variety of specimens, with chemical analyses of the most important by Professors Daniell and Wheatstone.

These valuable tables, with the report, are published in the Civil Engineer and Architect's Journal for September, October, and November of the present year, a very able and useful publication, to the editor of which the profession of engineers and architects are indebted for a vast accumulation of practical and scientific information.

In concluding the last of this series of practical papers, we propose to take a brief review of the construction of contracts for building bridges, in so far as they relate to the engineering of these works Any observations we shall make on this subject will apply equally to the contracts for constructing most other engineering works. In fact, this general application is necessary, as far as the bridges on railroads are concerned, because, in railway specifications, the bridges are included in the same contract with many other works; and, of course, the general clauses in the contract deed apply to the bridges in common with the other works.

The object of these general clauses is, in the first

place, to bind the contractor to execute certain works according to the designs and specification of the engineer. The designs are affixed to the contract in the shape of plans, sections, and general drawings, explanatory of the works to be constructed, and the specification follows the contract as a schedule, which is occasionally referred to as the technical instrument for guiding the operations of the contractor.

In the next place, the general clauses provide for any alteration in the original design of the engineer, and give to the latter an arbitrary power of directing the contractor to make any alterations which he (the engineer) may think proper. The way in which alterations, whether of increase or diminution in the amount of works, are to be estimated is usually regulated by a schedule of prices which the contractor is required to furnish as a part of his tender to execute the work, and this schedule is also appended to the contract deed.

Another series of clauses, usually introduced into railway contracts, confers upon the engineer an absolute power over the entire operations of the contractor, so that not only the method of construction, but also the times and seasons for working, the amount of labour to be employed, the tools and machinery to be used, are all subject to the approval of the engineer.

In order that all these clauses may be enforced for the benefit of the company, (who, as parties to these contracts, most assuredly take care to render their own the stronger side,) they are empowered by another clause, at any time they shall think proper, to set aside the con-

tract, to stop the contractor in his work, and take possession of all his tools, materials, and property of every kind which may be on the site of the works. By this act of seizure the company take the work into their own hands, and they may proceed to complete the same entirely independent of the contractor, from whom and from his sureties the company, after having executed the works under their own management, may recover any excess of cost which shall have been incurred over and above the original contract sum.

We shall now proceed, in detail, through the principal clauses of a contract deed for building a bridge.

Passing over the preamble, which is frequently a mere legal form and without interest to the engineer, the first clause sets forth, that in consideration of the sum or sums of money thereinafter agreed to be paid, the contractor, for himself, his heirs, executors, and administrators, doth covenant, promise, and agree that he will begin, and in a substantial, perfect, and workmanlike manner, and to the satisfaction, and according to the directions, of the engineer, build, erect, complete, and finish the bridge in the manner thereinafter mentioned, together with the approaches thereto, and all and every other work and works, according to the plans agreed upon by the parties to the contract, and under and subject to the directions, rules, regulations, explanations, and restrictions mentioned or referred to in the specification contained in the schedule thereunder written or thereunto annexed, with such alterations (if any) as from time to time shall be directed by the engineer.

The succeeding clauses go on to provide that the contractor is to furnish, at his own cost and charges, all the materials incident to or necessary for executing the works.

Then follows a clause which places upon the contractor the responsibility of the whole design, in case of failure, from any cause whatever. The clause in bridge contracts is frequently to this effect,—"That the contractor shall bear all loss, risk, and responsibility whatsoever attending the execution of the works hereby contracted to be performed, and shall and will, at his own expense, forthwith, and without any delay, make good all damages of every description, by floods or otherwise, which may happen to the said works, or any part thereof, during the progress of the same."

We would humbly suggest, that alike for the credit of the engineer, and in justice to the contractor, this clause ought to be modified wherever the former feels that confidence, which we presume he always ought to feel, in his own designs. When certain works are specified, and their mode of execution particularly detailed, and where the contractor has faithfully followed the instructions, and adhered to the specification of the engineer, he ought at the most to be answerable only for those accidents and casualties which arise from sudden visitations of the elements, such as damage by lightning, earthquakes, or floods.

The contractor's tender may be made sufficient in amount to cover all probable risk from accidents of this kind; but it is a more difficult affair to estimate the

chance of failure from a new or partially untried design; and if the engineer, who is himself the author of the design, should shrink from the responsibility of constructing it, with how much greater reason may the contractor plead the hardship of a position which renders him liable to such serious risk!

The next clause specifies the time by which the work is to be completed, and occasionally imposes a penalty, to be paid by the contractor, for each week that the actual time of execution shall exceed the time specified. The contractor is also very frequently bound to keep the bridge in repair for one, two, or three years after completion, in order that, in case of any failure from imperfect construction, the expense of restoration may be borne by him, as provided in a former clause. The contract sum is then set forth, and the times and mode of payment are either mentioned in this place or at the end of the contract.

The next clause relates to alterations in the original design, and is more or less severe upon the contractor in different contracts. One of the strictest forms we have seen, as giving to the engineer entire and absolute power with respect to alterations and payment to the contractor, provides, "That, in case at any time or times, during the progress of the works, the engineer shall think proper to cause any alteration in, or variation from, the original plans and specifications to be made, either by increasing the said works, or the scale or magnitude thereof, or omitting some part thereof, or diminishing the said works, or the scale or magnitude

thereof, or altering the quality of any part of the works, or the materials to be used therein, or other howsoever, the contractor shall and will execute, form, and complete the said works according to e or any such alteration or variation in the manner, within the time in which the said works ought t completed, according to the true intent and meani these presents, and no such alteration or variation vacate or lessen the validity of any of the cover or agreements herein contained, but such sum of m shall be added to or deducted from the contract as the engineer shall estimate to be the value of alteration or variation."

The contractor by this clause is evidently place the mercy of the engineer, with whom it is ent optional whether any and what allowance shall be r for what are usually termed extra works, and for e other kind of alteration which the engineer may ch to make. The form of the above clause is usually v in railway contracts so as to make the schedule of p the standard by which alterations or variations are t valued; but in many of these contracts it rests with engineer to determine whether any allowance at all be made to the contractor, or whether in lieu ther deduction is to be made from the contract sum. U either form of clause the power of the enginee absolute; and we apprehend, whatever may be the tegrity and general moral excellence of the man sessing such a power, it is too great for him to exe without the danger of partiality. We have no inte

of dwelling at length upon this particular subject; but let any unprejudiced mind consider on the one hand the engineer pledged to execute his work for an insufficient estimate, with the interest of his own employers to consult, his own credit at stake in many different ways, and on the other hand no possible, or at least no ostensible, object for favouring the contractor, and judge to what side his opinion will incline. Let the contractor in such a case value his chance at what he pleases, he will over estimate it; the final issue will probably be that he must content himself with having bought experience at a price which he can probably afford to pay only once in his life.

In the specification which follows the account of Hutcheson Bridge in another part of this work will be found the fairer and more reasonable stipulation with regard to extra works, which entitles the contractor to be paid for these upon the certificate of the engineer, according to the schedule of prices.

The next clause empowers the engineer to reject any materials brought upon the site of the works, and the contractor is bound to remove such rejected materials, or pay for the expense of removal by workmen to be employed by the engineer.

In a succeeding clause the engineer is further empowered, in case he should disapprove of the workmanship, materials, or execution of any part of the works, to order the contractor immediately to take down and re-execute or alter the disapproved part or parts to his (the engineer's) satisfaction. And should the contractor refuse

or neglect to comply with this order within the space of one week, the engineer may employ workmen to take down, alter, amend, or rectify such disapproved parts of the work. The cost of employing these other workmen, and defraying their bills for labour and materials, to be deducted out of the balance then due to the contractor; and should this balance be insufficient, the contractor or his sureties must make good the deficiency.

The clause by virtue of which the company may invalidate the contract and take the work into their own hands is variously modified in different deeds. As this clause is very important, we shall give the form of it in full.

" In case the contractor shall refuse or neglect to perform the (aforesaid) works, or any of them, in manner hereinbefore in the said specification mentioned, or to obey and comply with any orders or directions to be given by the engineer, or in case at any time during the progress of the works there shall appear to the engineer to be any unnecessary delay in the carrying on of the works, or any part thereof, either by not employing a sufficient number of workmen, or otherwise howsoever, or in case any of the works shall not be performed to the satisfaction of the engineer, or shall not be finished within the time hereinbefore mentioned for completing the same, then, and in such case, it shall be lawful for the company to revoke and make void this present contract, and every clause, matter, or thing herein contained, so far as the same relates to the part of the said works which shall not have been performed, and the

contractor shall be entitled only to such part of the said sum or sums of money hereinafter agreed to be paid as shall be estimated by the engineer to be the value of the part of the works which shall have been performed."

This clause, in the case of railway contracts, is rendered more severe upon the contractor, inasmuch as "the company, if they shall think fit so to do, may seize and take possession of the works, and of all or any part of the materials, engines, machinery, implements, and utensils provided by the contractor for the execution thereof, and may employ any other person or persons, either by contract, or measure and value, or otherwise, or by themselves, their engineers, servants, agents, workmen, and others, may proceed with and complete the said works according to the terms of the specifications, and the true intent and meaning of these presents, &c."

The contractor is frequently prohibited from making sub-contracts, or from under or sub-letting any part of the works without the consent of the engineer; but of course this particular prohibition is quite unnecessary wherever the engineer is armed with the arbitrary powers conferred by preceding clauses, and where the company may invalidate the contract whenever the work is not proceeding to their satisfaction.

As it is far from unlikely that disputes may arise relative to the meaning or interpretation of the drawings and specification, the engineer is appointed in case of any misunderstanding to construe, interpret, and explain; and again, in case of discrepancy or disagreement between the drawings and specification, the engineer's de-

cision as to which of these shall be followed is n
binding on the contractor.

In railway contracts there are usually some cla
of minor importance, such as that which obliges
contractor to make accurate returns to the engine
the number of workmen of all kinds employed on
work from week to week. In addition, also, ther
one which vests in the company, during the progre
the work, all the property of the contractor which
be brought upon the site of the works for the pur
of being used therein, and the contractor is prohib
from removing any part of such property except u
the sanction of the engineer or that of the company.

The contract concludes with the covenant, on the
of the company, that they will pay to the contracto
sum for which the works are to be performed at
times, and in such proportions, as are therein set for

Such is the usual form of contracts for bui
bridges in this country; and subject to stipula
differing slightly, or not at all, from those re
above, many of our largest works have been exec
The construction of the deed being commonly man
by gentlemen of the law, it is usual to find the su
matter very much amplified, and of course a great n
unnecessary words and phrases are employed; bu
preceding extracts and outlines of clauses may be t
as the real substance of the most complete contract
for the building of bridges.

T. H

SOME ACCOUNT

OF

HUTCHESON BRIDGE OVER THE CLYDE AT GLASGOW.

BY LAURENCE HILL, Esq., L.L.B.,

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Most of our readers are aware that among the rivers of Scotland the Clyde ranks next in importance to the Tay, which discharges more water than any other British river, and that Glasgow, to which the Clyde gives such great commercial importance, stands highest in the scale of population among the cities of Scotland.

Prior to the year 1772, when the Broomielaw Bridge was opened, the thoroughfare across the river at Glasgow was confined to the narrow bridge opposite Stockwell Street, built by William Rae, bishop of Glasgow, in 1345, which, after the Reformation, became the property of the corporation of that city.[1] This, like almost all ancient bridges, was incommodiously narrow,

[1] The *name* of the old bridge, and by which in some forgetfulness of the circumstances the trustees have recently designated the new bridge at Jamaica Street, is "The Bridge of Glasgow;" the first Act of Parliament, 32 Geo. II., for building the new bridge, expressly narrating the circumstance of the Stockwell Bridge being so called.

and its roadway steep. It has, however, undergone several alterations by which its breadth has been extended and its approaches greatly improved. From the imperfect state of the navigation between Greenock and Glasgow, and the rising importance of Paisley, the traffic between these places along the old bridge became so very inconvenient that the new bridge at the Broomielaw was found indispensable. But this bridge has in its turn become too narrow for its traffic, and has been removed and rebuilt in a much more spacious and elegant style. Between these two stone bridges a timber bridge was erected in the year 1831, agreeably to plans by Mr. Stevenson, engineer for Hutcheson Bridge, in a style accommodated to the flat nature of the banks, and with a level roadway. Higher up the river we have the recent erection called the "Hutcheson Bridge," situated immediately above the Court Houses, forming a direct *entrée* to the city from Ayrshire.

Hutcheson Bridge is so called from two brothers, George and Thomas Hutcheson, of Lambhill, the elder of whom died in 1640, and the younger in 1641, leaving large sums to endow an Hospital for the relief of the aged and infirm, and also for the education of youth. The magistrates and clergy of Glasgow were appointed trustees, and instructed by the donors to lay out the money "upon the best and cheapest arable lands that can be got near this burgh." In pursuance of this mandate the trustees purchased the half of the barony of Gorbals, which is now, perhaps, among the most important appendages of the city of Glasgow.

The feuars of Hutcheson-town were bound by their titles to pay an additional feu or ground-rent upon the erection of a bridge over the Clyde leading immediately into their lands, and the trustees for the Hospital, along with others, entered into contract with a Mr. Roberton to build the proposed bridge, to consist of five arches, for the sum of £6,000; the extreme length was to be 400 feet, and its width within the parapets 26 feet. The specification of the work, however, was unfortunately defective, and wanted solidity proportioned to the force of the current and depth of foundation suited to the soft nature of the bottom. The work, however, went on, and all the arches were turned, and the retaining walls nearly finished, when, on the 18th November, 1795, in one of those floods to which the Clyde is incident, the fabric was unfortunately swept away. Mr. Stevenson, the engineer of the present bridge, was at this time in Glasgow, and when the alarm was given he instantly repaired to the spot, where the river presented a sad scene of ruin. The arches had burst upwards with so much force that the key-stones were thrown to a considerable height above the arches. The northmost arch was carried away at about two o'clock in the afternoon, and before sunset the whole were gone, and the bed of the river immediately below the site of the bridge, and where the medium summer level, at low water, is only 18 inches, was then scooped out to a great depth, in some places even to 28 feet.

In a work treating of bridges, details of this kind

become interesting as matters of history, and profes-
ally valuable to the engineer. A question immedi-
arose between the contracting parties as to who
to be the losers in the business. The contractor
lost the labour of at least two years, and the tru-
had by this time advanced the sum of £4,000
account of the works. It was alleged, on the pa-
the contractor, that the magistrates, as trustees,
refused or delayed the erection of a retaining
connected with the abutment of the bridge on the
side, and the stability of the bridge was said to
been endangered by a large quantity of rubbish,
properly deposited at the northern abutment.
question as to the cause of the damage was at le-
submitted to the late eminent Mr. Rennie, engi-
and the late Mr. Wilson, architect, who reported
the subject, and the contractor was ultimately reli-
from his obligations on refunding the sums which
been advanced him, without interest.

After this accident nothing further was done tow-
the erection of Hutcheson Bridge for about twenty-
years. But the increasing inconvenience to the lan-
Hutcheson Hospital, and the public generally, ha-
been brought under the notice of the trustees for
Hospital, the proposal of a new bridge was wa-
espoused by Robert Dalgleish, Esq., as Precepto-
President of the Hospital, and the Very Rev. Prin-
Macfarlan, of the College, one of the committee,
always took a lively interest in the recompletio-
the meausre. It was accordingly finally resolved o-

the year 1828, a new Act of Parliament being expressly obtained for this purpose, and also for opening a new road or street in connexion with the bridge, through the Hospital's estate, to the great turnpike road between Glasgow and Ayrshire. Although the detailed specification of this bridge is given in this work, yet, viewing Hutcheson Bridge as one of the most considerable of Scottish bridge-works, a descriptive account of it is here given, for the use of those who may not have occasion to look into these technicalities. The design of this bridge was made by Robert Stevenson, Esq., of Edinburgh, and the works were contracted for and executed by Mr. John Steedman, now local engineer for the county of Donegal. As a piece of masonry, Hutcheson Bridge stands unrivalled; and in an engineering point of view it may be regarded as remarkable from the difficulties which attended the arduous works for establishing the foundations of the piers and abutments of the bridge.

This bridge consists of five arches, which are segments of a circle whose radius is 65 feet. Two of the arches are 65 feet, two 74 feet 6 inches, and the middle arch 79 feet, in the span; with versed sines of 8 feet 8 inches, 11 feet 9 inches, and 13 feet 4 inches, respectively. This rise is proportionally less than perhaps that of any other arches of the same extent and figure in the kingdom. By this means, although the banks of the river are low, approaches are obtained on either side with very little embanking or damage to the adjoining streets, and a line of draught is obtained at the easy rate of one in

thirty. Although, for our part, we are inclined to think the beauty and strength which this bridge possesses would make its own features and proportions better shown without the foreign aid of any ornament at all, yet the elevation, as will be seen from Plate No. 27, is adorned with columns and niches, the columns resting on the piers and supporting an appropriate entablature which runs over the arches throughout the whole length of the bridge. The niches were designed to contain statues of the benevolent founders of the Hospital on the one side, and figures emblematical of the objects of the charity on the other. The columns are so arranged as to be capable of supporting footpaths, should it be found advisable to widen this bridge, as has been the case with those further down the river. The carriage-way is macadamized, and the footpaths, on either side, are laid with Caithness pavement, which, besides being remarkable for cleanliness, is exceedingly durable.

The ceremonial of laying the foundation stone was performed by Robert Dalgleish, Esq., the Lord Provost of Glasgow and Preceptor of the Hospital, on the 9th day of October, 1829, and the works were completed in 1833. The contractor had unfortunately not only a succession of bad seasons to contend with while this work was in progress, which kept the river much in flood, but when the ground was opened for the foundations of the abutments and piers it turned out to be extremely untoward. When the foundation pit of the northern abutment had been excavated to the specified depth, its appearance was so unfavourable that the

trustees, in some alarm, sent for the engineer, to consider whether the excavations for the foundations were really practicable to the specified depth. The appearance of the bottom of the excavated part was no doubt most forbidding, the whole being in a state of motion, with perhaps a hundred springs of water boiling up. The soil in fact was what is called *running* sand, mixed with minute portions of silt. It was therefore found necessary to enlarge the platform for the foundation course of that abutment, to increase the number of bearing piles, and to drive the outer row of them close, like sheeting piles. It was at the same time determined not to draw the inner row of the sheeting piles of the coffer-dam, but to drive them home, as an additional security to the foundation. On the engineer's reporting the propriety of these extra and precautionary works, they were at once acceded to and ordered by the trustees.

Great difficulty was found in keeping under the springs of water which boiled up over all the foundation with great force, and also in the close driving of the extra piles, which often recoiled to the stroke of the piling machine. The foundations of the former bridge were likewise found very troublesome to remove. The piles, as will be seen from the annexed specification of the bridge-works, were 9 inches square and 18 feet long, driven to the depth of 9 feet under the level of the summer water mark; at which level the platforms were laid. The ground was found to be of much the same nature all the way across the river, though it rather improved toward the southern side. But here

also work of considerable expense was required, for t safety of the lofty houses of the contiguous street. became necessary to drive a row of piles landward the southern abutment of the bridge, as a circumv lation, as shown in Plate 29.

The best materials which the sandstone quarries the neighbourhood of Glasgow could produce were lected by Mr. Laidlaw, the resident inspector, wh were afterwards carefully dressed and built under superintendence. The mortar was prepared in l manner from the water-limestone quarry of Arden Renfrewshire, and from Aberthaw in Wales.

Hutcheson Bridge, on the whole, is believed to cont the best collection and the largest sized materials of a of our public works of similar extent. This bridge, be first in order, has the burst of the floods and ice to b and regulate for the City Bridge below; and from circumstances attending the former bridge, with the gineer's knowledge of the nature of the river, he well to provide for a massiveness and strength of spe fication, as indispensable to the permanency of str tures similarly situated. The engineer of the splen bridge at Bourdeaux, which Mr. Stevenson had carefu considered while it was in progress, and which prese various points of similarity, has constructed a gr extent of additional masonry, forming two archw within the walls of the bridge itself, chiefly for atta ing the same object.

Independently of the expense of the Act of Parliame engineering charges, and other contingencies, the contr

price and extra works of Hutcheson Bridge amounted to upwards of £23,000. And though the contractor did not find this a work of profit, yet with his perseverance and unflinching steadiness, in spite of many harassing difficulties, he succeeded in completing one of the best pieces of bridge masonry that has been executed in the kingdom.

SPECIFICATION

OF

HUTCHESON BRIDGE OVER THE RIVER CLYDE AT GLASGOW.

BY ROBERT STEVENSON, OF EDINBURGH,

CIVIL ENGINEER.

Edinburgh, April, 1828.

HUTCHESON Bridge is to be built across the River Clyde at Glasgow, in a line with Crown Street and the front of the Court House, agreeably to a design made by Robert Stevenson, Civil Engineer, Edinburgh, who, previously to the commencement of the work, will more precisely determine and point out the site and line of direction of the bridge, which is to be executed and completed as hereinafter described in this specification, and represented in the accompanying drawings, signed by him as relative hereto.

EXTENT.

The bridge is to consist of 5 arches, having an aggregate water-way of 358 feet; and including the 4 piers, the whole extent from the front of the abutment at the

impost course on the southern side of the river to the front of the abutment on the northern side is to measure 404 feet, independently of the abutments and landward wing walls on either side. The breadth of the bridge over the soffit of the arches is to be 38 feet. The elevation and works on the one side of the bridge are to be equal and similar, in all respects, to the elevation and works on the other side.

SUMMER WATER LEVEL.

The surface of the River Clyde being subject to considerable variation of level, from the joint effects of tidal water and land floods, its summer water level, when unaffected by the tide, is hereby fixed at 18½ feet below the arrow point cut upon the Court House near the south-eastern angle of that building, to denote the height of the water of the Clyde during the great speat in the year 1782, which mark is to be referred to in regulating the specified depth of the foundations and other dimensions of the bridge.

COFFER-DAMS.

It having been ascertained, by boring and mining, that the subsoils of the bed of the river consist of gravel, sand, and mud, to the depth of 27 feet and upwards, it becomes necessary to prepare foundations of pile-work for the bridge; and therefore, to ensure the proper and safe execution of the works, coffer-dams are to be constructed round each of the foundation pits of the two abutments and four piers, of such dimensions as to

afford ample space for driving piles, fixing wale-pieces, laying platforms, pumping water, and setting the masonry; and likewise for the construction of an inner or double coffer-dam, should this ultimately be found necessary.

The frame-work of the coffer-dams is to consist of not less than two rows of standard or guage and sheeting piles, kept at not less than 3 feet apart for the thickness of a puddle wall or dyke, which space is to be dredged to a depth of not less than 9 feet under the level of the summer water mark above described, before the sheeting piles are driven. The guage or standard piles are to measure not less than 24 feet in length and 10 inches square; they are to be placed 3 yards apart, and driven perpendicularly into the bed of the river to the depth of 16 feet under the level of summer water mark, thereby leaving 8 feet of their length above that mark.

Runners or wale-pieces of timber, 9 inches square, are then to be fitted on both sides of each row of the guage piles, to which they are to be fixed with two screw bolts, of not less than one inch in diameter, passing through each of the guage piles. One set of these inside and outside wale-pieces is to be placed at or below the level of summer water mark, and the other set within one foot of the top of each row of said piles; the whole to be fixed with screw bolts in the manner above described. The wale-pieces are to be 4½ inches apart, in order to receive and guide the sheeting piles. This is to be effected by notching the wale-pieces into the guage piles.

The sheeting piles are to be 21 feet in length, $4\frac{1}{2}$ inches in thickness, and not exceeding 9 inches in breadth. They are to be closely driven, edge to edge, along the space left between the walings, and each compartment of the sheeting between the guage piles is to be tightened with a key-pile.

The coffer-dam frames are to be properly connected with stretchers and braces before commencing the interior excavation. Each coffer-dam is to be provided with a draw-sluice, 14 inches square in the void, with a corresponding conduit passing through the puddle dyke at the level of summer water mark. To render the coffer-dams water-tight, the whole excavated space between the two rows of piling is to be carefully cleared of gravel, sand, or other matters, to the specified depth, and clay, well punned or puddled, is then to be filled in and carried up to the level of the top of the sheeting piles. But if it shall, notwithstanding, be found that the single tiers of coffer-dam do not keep the foundation pits sufficiently free of water for building operations, the water must either be pumped out and kept perfectly under by steam or other power, or else excluded by the construction of a second tier of coffer-dam, similar in construction to the first.

For the foundation pits of the two abutment piers on either side of the river it is not expected that more will be required on the landward side for keeping up the stuff than a single row of guage and sheeting piles; but if the engineer shall find other works necessary, upon opening the ground, they must be executed by the con-

tractor, and shall be paid for agreeably to the cont schedule of prices for the regulation of extra and s works.

PILE-WORK OF FOUNDATIONS.

The stuff within the coffer-dams is to be excav to the depth of 10 feet under the level of summer w mark for each of the piers, and 8 feet for each of abutments. Bearing piles are then to be driven straight rows, longitudinally, over the whole exten the space to be occupied by the foundation cours each of the piers, and also of each of the abutme with their wing walls. These bearing piles are to b beech or Memel timber, of straight growth, measurin feet in length, and not less than 9 inches square. T are to be hooped at the top and shod at the bottom iron, of dimensions to be approved of by the engi (after the ground has been opened), and driven their e length into the ground, or as far as they can be dr by means of a cast-iron ram, weighing not less tha cwt., falling through a space of 30 feet, until they do sink more than one inch upon a *tally* of 10 strokes such other proof as may be satisfactory to the engi These piles are to be pitched at distances not gr than 2 feet 9 inches apart. The site of each abutm with its wing walls, is to contain not fewer than bearing piles, and of each pier, not fewer than 84. in the event of the ground proving softer when op than is anticipated, it may be found necessary to c extra piles, so that the outer row may be driven c

after the manner of sheeting piles; and also to increase the number of interior bearing piles according to circumstances. Such extra piling is to be paid for agreeably to the contract schedule of prices for the regulation of extra and short works.

WALE-PIECES.

After the bearing piles have been driven to the specified depth and number required, their tops are to be brought to one level, and the stuff being removed from between the pile heads to the depth of 1 foot 6 inches, a runner or wale-piece of beech timber, 12 inches in depth and 6 inches in thickness, is to be fixed at the top of the outward row of bearing piles all round the site of the foundations, with two screw bolts of one inch in diameter passing through the wale-pieces and the heads of each of the bearing piles in the exterior row, or, in the event of close driving, at distances not greater than 3 feet apart. In the same manner three stretchers of the same dimensions and description of timber are to be placed transversely, on a level with the top of the bearing piles, and fixed at their ends and middle to the piles, thus dividing the site of each foundation into four compartments.

UPFILLING BETWEEN THE PILES.

The spaces between the heads of the bearing piles, all over the site of the foundations, are to be made up with Whinstone road metal, broken so that each piece in its greatest dimensions shall pass through a ring 2½

inches in diameter, laid in strata not exceeding 6 inches in thickness, and beaten all over with a pavier's beater, weighing not less than 60 pounds; each layer to be well grouted with mortar prepared agreeably to directions under the article "Mortar," and the whole, when finished, to be on a level with the pile heads.

SILL-PIECES.

Sill-pieces of beech timber, measuring not less than 12 inches by 7 inches, are then to be laid longitudinally upon the site of the foundations and tops of the piles, to which they are to be fixed down with oaken trenails 1½ inch in diameter, and 21 inches in length. Timber of the same scantling and quality is in like manner to be fastened to that part of the piling which is under the cutwaters of the piers, and along the ends and sides of the abutments and wing walls, and is to be half checked at the ends of the sills, and secured to them by oaken trenails.

UPFILLING BETWEEN THE SILL-PIECES.

The spaces between these timbers or sill-pieces are to be built up flush with their upper side by *one* course of square dressed stones of a size sufficient, both in breadth and depth, to fill up the spaces. These stones are to have a chisel-draught round the edges; they are to be pick-dressed, and set in mortar.

PLATFORMS.

When the foundations of the piers and abutments,

and so much of the wing walls as may be found necessary, have thus been brought to an uniform level surface, a platform of beech timber 3½ inches thick is then to be laid down transversely, each plank of which is to be equal in length to the full breadth of the foundation at the place where it is laid, and is not to exceed one foot in breadth. The planks are to be set flush in mortar, and fixed to each sill-piece by two iron spikes, measuring one half inch square at the neck and 8 inches in length.

ABUTMENTS AND PIERS.

The first course of masonry of each abutment is to be laid on these platforms at the depth of 7 feet under the level of summer water mark, and is to measure over all 49 feet in length and 17 feet in breadth between the face next the river and the soffit of the crown of the horizontal arch. The first eourses of the masonry of the piers is, in like manner, to be laid at the depth of 9 feet under the level of summer water mark, and is to measure 53 feet 6 inches in length between the extreme points of the cutwaters, and 17 feet 6 inches in breadth. The foundation of each of the abutments is to consist of five courses of squared masonry, of the average thickness of 16 inches, and having an offset of 6 inches all round. The foundations of the wing walls are to have but two offsets on the inward face, and are to be finished in all respects agreeably to the drawings. The foundations of the piers are in like manner to have offsets of 6 inches on each of the first six courses, which are to average 16 inches in thickness.

The height of each of the piers, from the highest offset to the top of the impost course, is to be 10 feet 6 inches, with a batter or slope of 3 inches all round. The breadth of each of the middle piers immediately above the offsets is to be 12 feet 6 inches, and at the top, exclusive of the projection of the imposts, it is to be 12 feet, while the breadths taken at the same place on each of the landward piers are to be 11 feet 6 inches and 11 feet respectively. The shafts of the two middle piers are each to measure 47 feet 8 inches between the extreme points on the level of the highest offset, and on the level of the impost course 47 feet 6 inches, while the dimension at the same places in each of the landward piers is to be 46 feet 6 inches and 46 feet respectively.

MASONRY.

The masonry of the abutments and piers is to be of squared work. Each course is to be of the same thickness throughout, and, after the stones are set, the course is to be carefully brought to one level before proceeding with the next course. To suit the quarried materials, the courses above the foundations may vary from 12 to 16 inches in thickness, the face-work being composed of a course of headers and stretchers alternately. The header stones are to have bond into the wall of not less than 3 feet 6 inches, and are not to be less than 2 feet in length of outward face. The stretchers are not to be less than 2 feet in breadth on the bed, and 3 feet 6 inches on the outward face. These courses are to be laid so

as to break joint properly, as will be hereinafter more particularly described.

HORIZONTAL ARCHES OF ABUTMENTS.

The back or landward side of the abutments is to be built in the form of a horizontal arch of an elliptical form, which is to spring from the wing walls. These walls are to be built in regular courses, forming a continuation of the masonry of the abutments. The stones of the exterior faces or soffits of these arches are to be laid in courses of header and stretcher alternately, the headers not being under 3 feet in length, and 1 foot 9 inches in breadth of bed, and the stretchers not under 1 foot 6 inches in breadth of bed, and 3 feet in length of face. They are to be worked in the beds and joints with chisel-draughts round the edges, and pick-dressed between. The horizontal arches are to set off from the foundation course as already described, and to be carried upwards by offsets in common with the abutments. Above these offsets the same arched form is to be carried up perpendicularly to the level of the top of the springing course, where there are 3 more offsets of one foot each. Above this there is a course of flag-stones at the same height as the covering stones of the spandril walls, afterwards to be described. The stones of the horizontal arches are to average 2 feet 6 inches in depth of bed at the springing of the arch, and 2 feet at the crown. They are not to be less than 2 feet 6 inches in length, to be built in vertical bond, and

closely jointed with the hearting of the abutment. The bed joints are to be dressed in a similar manner to those of the face-work of the wing walls, the outward face to be chisel-draughted round the edges for accuracy in setting, but the space between to be left undressed. The other dimensions to be taken from the plans.

WING WALLS.

The beds, joints, and face-work of the wing walls are to be dressed in a similar manner to those of the horizontal arches. The hearting to be similar to that of the abutments. The dimensions of the different parts are to be taken from the written dimensions on the plan. In the opening behind the horizontal arches, and for 10 feet beyond the ends and sides of each of the wing walls, excavated matters from the foundations, mixed with gravel and a proper proportion of clay to give tenacity, are to be filled in to the top of the wing walls. This mixture is to be laid down in strata corresponding as nearly as may be with the courses of the building, each stratum being well beaten down as the courses are laid.

DRESSING OF STONES.

The stones are all to be hewn or worked to the full size specified. The beds droved round the outward edges to the breadth of 3 inches, and broached with mallet and iron within these draughts. The outward faces of the courses below the level of summer water mark are to have 1½ inch chisel-draughts round the

edges, and are to be pick and hammer-dressed between; above the summer water mark the face-work of the abutments and piers is to be neatly broached, and the horizontal joints chamfered to the breadth of 1 inch on the beds and faces. The hearting stones of the abutments and piers are, in like manner, to have $1\frac{1}{2}$ inch chisel-draughts round the edges of the horizontal beds and are to be broached between, but the vertical joints are to be dressed square with the pick or hammer. The impost courses are to be 15 inches thick, worked with a fillet of 3 inches in thickness, and a torus of 12 inches, projecting 8 inches beyond the face-work, as represented in the drawings. The cutwaters of the abutments and piers are to be built in courses along and in correspondence with those of the main body of the work, and finished with a continuation of the impost, surmounted by a blocking course of 2 feet 3 inches in thickness, formed for the support of the columns, as shown in the drawings.

The springing course of the arches is to be 4 feet in thickness. On the piers it is to be laid in three rows of stone; the two outer rows forming the springing stones are to be worked off in the face so as to form part of a voussoir having a soffit of 9 inches in breadth, and a bearing or breadth of bed on the pier of 3 feet 6 inches; and no stone is to be less than 2 feet 6 inches in length of face. The middle or closing row of the springing course is to be laid in two courses of such a breadth as to fill exactly the space left between the springing stones.

The springing course of the abutments is to be in all

respects similar to that of the piers, and the space between it and the back of the horizontal arch is to be filled up with square jointed hearting stones in two courses, so as to fill up the space exactly. The whole of the stones of those courses are also to be carefully worked in their bearing joints, and, when laid, are to be grouted with mortar.

The beds of the voussoirs are to be droved round the edge to the depth of 3 inches, and broached between these draughts. The end joints to be chisel-draughted, and broached off between; the face-work of the soffit of the arch is to be neatly broached, and the bed joints across the arches, and the heads of the ring courses, are to be chamfered to the breadth and depth of one inch on each side of the face or soffit. The heads of the stones of the ring courses are to project 1½ inch beyond the retaining walls, as shown in the drawings. The back or crown of the voussoirs is to be worked off in the curve of the extrados to the breadth of 18 inches, so as to form a seat for the Aisler work of the retaining walls.

CENTRE FRAMES.

Trussed centre frames, for building not fewer than three arches, are to be properly framed and constructed of Memel timber, and elm, or other hard wood, thoroughly connected with mortise and tenon joints, screw bolts, and iron straps, agreeably to a model to be prepared under the direction of the engineer. These centre frames are to be placed at distances not exceeding 4 feet apart, properly supported on beams

resting wholly on the scarcements of the abutments and piers, and covered with planks of 3 inches in thickness, on which the arches are to be turned. It is in the power of the engineer, however, to direct that five sets of centre frames, or one for each arch, shall be prepared for building the bridge; the contractor being paid for such extra centres.

ARCHES.

The five arches of the bridge are to be segments of a circle of 65 feet radius. The bottom of the several springing courses is to be upon one level, and to be 9 feet 6 inches above the summer water mark. The span of the middle arch is to be 79 feet, and its rise or versed sine 13 feet 6 inches. The two arches next to the middle one are to be each 74 feet 6 inches, and their rise 11 feet 9 inches, and the remaining two arches next the abutments are to be each 65 feet in the span, and to rise 8 feet 9 inches.

The width of the bridge, measuring across the soffit of the arches, as before noticed, is to be 38 feet. The voussoirs or arch-stones are to measure not less than 3 feet in length across the soffit,—their depth at the key-stone or crown is to be 3 feet 6 inches, increasing as they approach the springing courses, according to a radius of curvature of 70 feet for the extrados, making the depth of the voussoir at the springing of the central arch 4 feet 6 inches, and that of the other arches proportionally less at the springing course, according to the above radius of curvature.

RETAINING WALLS.

The retaining or outward spandril walls on each side of the bridge are to extend in a longitudinal direction 1½ inch within the heads of the ring course, to which they are to be neatly fitted, and carried to the height of the curve coinciding with the crown of the arches. They are to be of the thickness of 3 feet at their foundation on the top of the springing courses, and to diminish to 2 feet 6 inches at the top. At the height of 7 inches under the range of the curve of the arches, a scarcement or offset of 9 inches in breadth is to be formed, as a rest for the covering pavement of the interior spandril walls.

The face-work of the masonry of the retaining walls is to consist of regular courses of Aisler work, neatly broached, laid header and stretcher. The headers are not to be less in the face than 2 feet in length, nor more apart than 9 feet in each course, nor shorter than the full thickness of the wall where they are laid. The stretchers are not to be less than 3 feet in length, nor 1 foot 4 inches in breadth. They are to be dressed square with mallet and iron on the beds, with a droved draught round the edge of 1½ inch in breadth, and broached between, and the ends dressed square at least 6 inches back. The courses of Aisler work are to range from 15 to 12 inches in thickness, becoming gradually thinner toward the top of the walls; each course is to be chamfered in the face on the lower and upper beds, of the same dimensions as those specified for the piers. At the junction of the face courses with the face courses of

the arches, the stones are to be worked to a circular form, and cut square at the joint to the depth of 3 inches, so as to give them the requisite strength ; the corresponding stone of the succeeding course locking into that immediately below. The backing of the retaining walls is to be of coursed hammer-dressed masonry, corresponding in thickness with the Aisler course, and properly bonded with it. Circular holes of 3 inches in diameter are to be cut through the retaining walls communicating with the voids between the arches, in positions to be afterwards pointed out, for the admission of air, and the escape of any water which may percolate through the roadway of the bridge.

COLUMNS AND PILASTERS.

Over the projections on each side of the abutments there is to be placed one square pilaster, and on each of the piers two columns executed in a plain Grecian Doric style. The columns on the piers of the middle arch are to measure 13 feet in height, including the capital, and 2 feet 4 inches in diameter at the base. Those on the two landward or outer piers are to be 10 feet in height, including the capital, and 2 feet 2 inches in diameter at the base. The pilasters on the abutments are to be 7 feet in height, including the capital, and 2 feet 4 inches by 2 feet. The columns and pilasters are to be set with their face line at the base 2 feet beyond the retaining wall. They are to be built in courses not exceeding 2 feet in thickness, and properly bonded with the Aisler of the retaining wall. The columns and

capitals are to be wrought agreeably to an enlarged drawing, which will be given to the contractor. The architrave on the columns is to be 1 foot 6 inches in depth, worked with a torus or band 1¼ inch in depth, and projecting one half inch from the face of the architrave; the breadth of which is to be made up of two stones, each of them being the full length and depth of the architrave. The frieze over the architrave is to be 1 foot 6 inches in depth or thereby at the lower end, corresponding with the frieze of the general elevation: this frieze is also to be of one stone, 1 foot in breadth, with closers to fill up the returns at the ends. It is to be backed with square dressed stones, laid as headers, extending to the inward face of the retaining wall, and equal in height to the frieze. Between the columns, on each of the centre piers, a semicircular niche is to be formed, 4 feet in width, and 10 feet 6 inches in height; the retaining wall at those parts being thickened behind to admit of the niche. The columns, architrave, and frieze are to be chisel-draughted in the beds to the breadth of 3 inches, and broached off with mallet and iron between. The face-work of the whole, including the soffit of the architrave, the niche, and the whole space between the columns, is to be executed in the best style of droved work, set in mortar, lipped to the breadth of 3 inches with Roman cement, and carried up along with the face-work and bonded with it.

SPANDRIL WALLS.

Spandril walls are to be built between the outward

walls, parallel to them and butting against the horizontal arches of the abutment at either end of the bridge, and dividing the whole included space into eight equal compartments. These walls are to be 2 feet in thickness at the bottom, diminishing to 1 foot 6 inches at the top, which is to be at the level of 14 inches under the top line of the retaining walls. After being levelled, the spandril walls are to be coped with a course of hammer-dressed flag-stones, 1 foot 6 inches broad and 7 inches thick.

TYE-WALLS.

Upon each of the piers a tye-wall, of 3 feet in thickness throughout, is to be built across the bridge, and is to be carried up and bonded with the retaining walls. These spandril and tye-walls are also to be properly bonded with each other, and built flush in mortar: holes 6 inches square are to be formed in each of the interior spandril and tye-walls, to preserve the circulation of air and drainage of moisture.

FRIEZE.

The frieze course, which is to be laid on the top of the retaining walls, as above described, is to be 1 foot 6 inches in height, and to project 2 inches over the face of the retaining wall. The stones of which the frieze is to be built are not to be less than 3 feet 6 inches in length of face, 1 foot 6 inches in height, and 1 foot 2 inches in breadth of bed. They are to be worked with a droved draught of 5 inches in breadth in the lower bed, and 3 inches in the upper bed along the edges

next the face, and with a common chisel-draught round the other edges of the bed: the space between is to be broached, the end joints to be cut square to the depth of 6 inches, and the face-work neatly droved; the whole to be set in mortar and lipped with oil putty. The frieze course is to be backed with good rubble masonry, to the depth of 2 feet 8 inches from the face, and set in mortar.

CORNICE.

The cornice, 1 foot 2 inches thick, and projecting 1 foot 3½ inches from the frieze, is to be returned round all the projections over the columns and pilasters. The cornice is to be moulded agreeably to the enlarged diagram on the plan: no stone in it is to be under 3 feet 6 inches in length, or 3 feet in breadth on the lower bed, so as to have a bearing on the frieze course of not less than 1 foot 8½ inches. The tails of the stones are to be checked down on the upper bed to a depth sufficient to admit the footpaths at their proper level. The stones are to be chisel-draughted round the beds to the breadth of 2 inches, and broached off between the draughts: the end joints are to be neatly droved to the depth of 1 foot 9 inches from the extreme projection of the moulding, and the whole of the moulded part is to be neatly droved. The cornice is to be set in mortar, and the projecting parts of the joints carefully grouted and pointed with oil putty.

COVERING PAVEMENT.

Over each of the eight spaces between the spandril

walls, flag-stones, or covering pavement, in a hammer-dressed state, are to be set flush in mortar, for supporting the carriage roadway and footpaths. These flag-stones are each to contain not fewer than 8 superficial feet of rock, and are to be 7 inches in thickness, laid with a bearing of not less than 6 inches on the retaining walls, and 9 inches on the spandril walls. On this pavement, to the full width of the bridge between the retaining walls, clay puddle is to be laid to the depth of 12 inches, in strata 6 inches thick, which is again to be covered with a stratum of gravel, free of earthy particles, to the depth of 4 inches in the middle, diminishing at the sides to 2 inches.

PARAPET WALLS.

The parapet walls on each side of the bridge are to be built of solid parapet Aisler masonry; the two parapet walls to be equal and similar to each other in all respects. The basement course is to be laid on the cornice course. It is to be 1 foot 2½ inches in thickness, 12 inches in depth, and the stones are not to be less than 4 feet in length. The dado of the parapet is to be 12 inches thick, built in two courses, each 13 inches in height, the stones of which are to be laid in lengths of not less than 4 feet, and set in such a manner that the base may project one inch toward the outside. The coping course is to be 11 inches in thickness, and 1 foot 2 inches in breadth, projecting both outwards and inwards similar to the basement course, and laid in lengths of not less than 4 feet, the whole height of the parapet wall being 4 feet 1 inch from the upper surface

of the cornice course. The parapet walls are to be enlarged to the extent of the respective projections of the entablature, both in length and in breadth, and their base, dado, and cope, are respectively to be formed round these projections. These breaks or blockings are to be built with square dressed stones, corresponding with the several courses of the parapet wall. The stones throughout these walls are to be worked in the beds similar to those of the cornice and frieze. The cope and base are to be neatly droved, and the dado finely and closely broached. The coping of the parapet walls is to be chamfered on the upper bed from the middle line down to 9 inches from its lower bed on both sides. The stones of the coping are to be secured to each other by a cast-iron dowel 1½ inch in diameter and 6 inches in length, let into each of the vertical joints; to be properly grouted and run up with Roman cement.

LAMP IRONS.

Six lamp irons, formed agreeably to the enlarged drawings, are to be erected on each of the parapet walls of the bridge, and are to be connected with a train of gas pipes communicating with Adelphi and Clyde Streets. These lamp irons and gas pipes are to be provided at the expense of the trustees, but are to be fitted up at the cost of the contractor, agreeably to the directions of the engineer.

FOOTPATHS.

A footpath of 5 feet in breadth, including the curb-

stone, is to be formed on each side of the bridge. For this purpose curb-stones are to be laid at the distance of 5 feet from the basement course of the parapet walls, the upper surface of the curb being 1 inch under the level of the said basement. These curb-stones are to be 1 foot 6 inches in depth, 9 inches in thickness at bottom, sloped to 6 inches at the top, and laid in lengths of not less than 2 feet. The spaces between the parapet walls and curb-stones, on either side, are to be laid with pavement on a solid bed of concrete, consisting of screened gravel mixed with one-fifth part of lime. The pavement to be neatly chisel-draughted round the edges, and the vertical joints squared, lipped, and pointed with Roman cement. The stones of the footpath are to measure not less than 2 feet in breadth, 4 feet 6 inches in length, and 6 inches in thickness, laid in mortar flush with the curb-stones at the one edge, and on a level with the bottom of the parapet walls on the other.

DRAINS.

Along the under side of the curb-stones, on each side of the bridge, and 9 inches under their upper edge, sky-drains or gutters of Whinstone rock are to be formed. The stones in these drains are to be not less than 1 foot 6 inches in length, 1 foot 2 inches in breadth, and 9 inches in depth, having the groove or channel wrought to a segment of a circle, 7 inches in width by 2½ inches in depth. By these gutters the drainage water is to be carried off the roadway into sewers communicating with the river.

ROADWAY.

The carriage roadway is to be metalled to the full extent between the sky-drains and along the whole length of the bridge, and formed with a line of draught rising on the one side and falling on the other at the average rate of one perpendicular to thirty horizontal, with a transverse section rising 3 inches in the middle. The road is to be laid to the depth of 12 inches with broken Whinstone, each piece of which, in its largest dimensions, is to pass through a ring of 2¼ inches in diameter. The whole to be consolidated by being repeatedly passed over by a rolling machine of not less than 1½ ton in weight.

DESCRIPTION OF MASONRY.

The face or outside work of this bridge is to be built of Aisler masonry, as has been already more particularly specified and described. The stones are to be laid throughout the whole works on their natural beds, or as they lie in the quarry. The arch-stones are to break joint or overlap each other not less than 1 foot. In all other parts of the face-work the vertical or perpendicular joints of the one course are to overlap the joints of the course immediately below not less than 9 inches. The whole of the masonry of the bridge is to be set flush in mortar, prepared as described under that article. The joints of the other parts of the masonry are to be carefully grouted with mortar, and the face-work neatly pointed. The beds of the face-work of the abutments and piers, to the height of the springing course, are to

be lipped or laid to the depth of 3 inches with Roman cement, and the beds of the face-work of the retaining walls, the pilasters with their basements and entablature, the cornice, parapet walls, and other exposed parts of the work, are to be lipped to the same depth with oil putty prepared with a mixture of fine sand and tinted of a stone colour, to the satisfaction of the resident engineer.

MORTAR.

The mortar for the foundations of the work and abutments, with their wing walls, the piers and arches, the retaining, spandril, and parapet walls, and other parts of the bridge, is to be composed of well burned limestone, of quality approven of by the engineer. It is to be burned at or near the works, and slaked as it comes from the kiln with pure river water; it is then to be mixed with sand, free of earthy particles, in the proportion of one measure of lime to two measures of clean sharp sand, or fine screened gravel suitable to the species of work for which it is to be used. This mixture is then to be worked to a proper consistency by manual labour, or a horse-gin and mortar-rollers if required. But, notwithstanding the proportions of lime and sand being above specified, these are to be subject to alteration on the inspection of the mortar by the resident engineer, who is also to decide upon the quality of the Roman cement and oil putty for lipping and pointing the works.

QUARRIES.

The stones for the outward walls or face-work, in-

cluding the shafts of the piers and face-work of the abutments, wing walls, arch-stones, retaining and parapet walls, including the frieze and cornice of the bridge, are to be taken from the best and hardest rock of such of the following quarries as may ultimately be approven of by the engineer as being capable of producing stones of sufficient dimensions, and of proper quality: viz. Garscube, Balgray and Woodside, in the vicinity of Glasgow, Nitshill in Renfrewshire, Carbrook in Stirlingshire, or Humbie in Linlithgowshire. For all interior work, such as the hearting of the abutments and piers, backing of the wing walls and rubble work in general, together with the covering pavement, the stones may be taken from the best and hardest rock of any of the quarries above specified; but if it shall meet the convenience or views of the contractor to take the whole of the materials from the quarry or quarries ultimately approved of for the face-work by the engineer, the contractor shall be at liberty to do so, and the whole of the materials may thus come to be procured from such quarry or quarries, provided always, that it be without detention or disadvantage to the works: a provision which the contractor will keep in view in any engagement he may be disposed to enter into with the quarry masters.

The contractor must keep a mason in the principal quarry from which the materials are to be supplied, whose duty it shall be to make careful selection of the stones; but independently of this condition, no stone will be permitted to be laid in any of the abutments,

piers, arches, or in any part of the face-work of the bridge, having what are called *drys* or *cutters*, or any unsoundness in its general aspect or appearance; or which from colour or want of hardness may appear doubtful to the resident engineer or said chief engineer, who may cause such faulty stone or stones to be removed, though actually built into the work.

The curb-stones, sky-drains, and road metal shall be taken from one of the quarries in the neighbourhood of Glasgow, which shall be approven of by the engineer; and the foot pavement from Castle-hill quarry in Caithnesshire, or any others of approved quality.

PROGRESS OF THE WORKS.

By the term of Martinmas, in the year 1828, the contractor is to complete the north abutment, with its wing walls, and three of the piers, to the level of the springing courses. By the term of Martinmas, 1829, the contractor is to complete the remaining abutment and pier, to turn the five arches of the bridge, and carry the retaining, spandril, and tye-walls to the height of 5 feet above the springing course: and by the term of Lammas, 1830, he is to complete and finish the whole of the remaining works of the bridge, hereinbefore specified and represented in the plans and drawings, signed by Robert Stevenson as chief engineer, and by the contracting parties as relative hereto.

WORK-YARD.

The contractor shall enclose the work-yard to be

provided by the trustees for their works, extending to two acres or thereby, with a wooden paling: he is also to erect a work-yard lodge, consisting of two floors, agreeably to a plan approved of by the engineer. The ground-floor is to be for the use of the contractor, and the upper-floor for the use of the resident engineer and trustees. When the bridge works are completed, the contractor is to remove the said paling and work-yard lodge, (the materials being his property,) and to restore and leave the ground as nearly as may be in its present state, by the removal of all rubbish and chips of stones, to the satisfaction of the trustees.

GENERAL OBLIGATIONS ON THE PART OF THE CONTRACTOR.

The contractor is to provide and pay for all timber and building materials, and for their carriage; he is also to pay the wages of all artificers and labourers employed in the work,—to provide a service bridge to facilitate his operations,—to furnish the pile-work for the foundations,—and provide three trussed centres for building the arches, with all their necessary supports and apparatus: he is also to furnish pumps, engines, cranes, tools, utensils, and implements of what kind or description soever as may be found necessary for preparing the foundations of the bridge, and executing and completing the whole superstructure, including its roadway and approaches, to the full extent of the bridge and its wing walls, in a safe, effective, substantial and workmanlike manner, under the direction and to the satisfaction of

Robert Stevenson, civil engineer; failing whom, to the satisfaction of any other civil engineer appointed by the trustees. The whole works are to be fully and completely executed at the sight of a resident engineer or inspector, who shall be approven and recommended by the chief engineer.

The contractor for these works shall not be at liberty to sublet any part of them without the consent of the chief engineer, excepting in so far as regards the works of excavation, quarrying stones, or the carriage of materials. The contractor shall find security for the due execution of the work and completion of the contract to the satisfaction of the trustees.

GENERAL OBLIGATIONS ON THE PART OF THE TRUSTEES.

The trustees are to provide the land for the site of the bridge, a work-yard of not less than two acres, as contiguous to the site of the bridge as may be found expedient, with the necessary access to the works, over and above the contract price for the bridge: the said trustees are to pay the contractor the sum of £ for enclosing the work-yard, and upon all building materials and implements laid down for the purposes of the work they shall have a right of property. The trustees shall also pay the contractor £ toward the erection of a work-yard lodge. They shall also convey to the contractor all the privileges of their Act or Acts of Parliament in regard to the conveying of materials for the work, free of toll or turnpike dues; they shall like-

wise pay the salary of the resident engineer, and cause the contract price for the bridge to be paid to the principal contractor, or his order, by instalments, proceeding upon the certificates of the engineer, or agreeably to his directions, in manner following: viz.

One-twentieth part of the said contract price when a quantity of materials of the value of £100 sterling are laid down upon the ground, and the deed of contract signed by the contractor.

One-twentieth part further when each of the platforms of the abutments and piers are respectively laid.

One-twentieth part when the first abutment is brought up to the springing course.

One-fortieth part for each of the piers and remaining abutments, when they are brought to the height of the springing course.

One-fortieth part when the arch-stones of each of the five arches have been worked, or when a quantity of arch-stones equal to each arch are hewn, and ready for being set.

One-fortieth part when each of the centre frames for the middle and two of the side arches shall have been prepared, and are ready for the work.

One-twentieth part when the retaining, spandril, and tye-walls shall have been carried to the full height.

One-twentieth part when the covering stones, frieze, and cornice, are laid.

One-twentieth part when the parapet walls and roadway of the bridge are completed. And the balance remaining of the contract price when all the works are

completed, and the bridge finally taken off the hands of the contractor by a letter under the hands of the engineer.

Should it so happen that the foundations or any other part of the works of the bridge are required to be carried to a greater depth and extent than is herein described and represented in the accompanying plans and sections of the work, the contractor, upon the certificate of the engineer, shall be allowed an additional sum in proportion to the extent of extra work done. But it is expressly declared and understood that any such extra works are to proceed only on the previous orders given under the hands of the engineer, or by his order, otherwise such extra charge shall not be sustained. If, on the other hand, any of the foundations or other works shall be diminished in extent, a rateable and proportionable deduction shall be made on the contract price, agreeably to the schedule and rates for additional and short works filled up by the contractor, with specific prices, and presented by him along with his offer for the contract, to which that schedule is hereby declared to form a relative paper.

It is understood that the trustees shall not be bound to accept the lowest offerer, but shall have the choice of the contractor from among the offerers.

CONCLUDING CLAUSE.

Should any misunderstanding arise as to the meaning and import of the foregoing specification or relative drawings, or about the quality and dimensions of mate-

rials, or the due and proper execution of the works, the same shall be explained and decided by the said chief engineer, whose award in all cases shall be final and binding upon the contracting parties. It is further declared and provided for, that if it shall at any time appear to the said chief engineer that the works are not proceeding in a proper and satisfactory manner agreeably to the contract, he shall have the power of discharging the contractor, taking the contract out of his hands, and employing others to finish the works. In the event of its being necessary to resort to such measures, the trustees shall pay such sums as shall be awarded by the engineer to the contractor for work done; but if it shall appear that the balance of the contract price is not sufficient for completing the remainder of the works, the difference, so ascertained and determined, shall be made good by the contractor; it being the full meaning and understanding of the contracting parties that the agreed upon contract price shall complete the whole works herein specified, and that the annexed schedule of prices refers only to extra works and deductions, proceeding upon such written orders for their execution as may be issued from time to time under the hands of the engineer.

When the works shall have been found by the engineer to have been duly executed and completed in all respects agreeably to the contract, and to his entire satisfaction, the same shall be intimated by a writing under his hands, within six months after the date of the same having been so declared. He shall then take the bridge

finally off the hands of the contractor, if the same shall then be found by him to be in good and substantial order. The balance of the contract price shall then be paid, and all further responsibility relative to the works removed from the contractor.

Schedule of rates of prices, including labour and materials, for ascertaining the value of extra and short works, or of alterations, referred to in the foregoing specification of Hutcheson Bridge, Glasgow, by Robert Stevenson, Civil Engineer.

	£.	s.	d.
1. Excavating foundations above the level of summer water mark, including removal, not exceeding 50 yards, shall be valued at per cubic yard			
2. Excavating or dredging foundations under the level of summer water mark, including removal, not exceeding 50 yards, at per cubic yard			
3. Memel timber or red pitch pine, in guage piles, independently of its value as old or used timber, at per cubic foot			
4. Sheeting piles, in like manner independently of the old timber, at per cubic foot			
5. Beech or Memel timber in bearing piles, sills, wale-pieces, and planking, at per cubic foot			
6. Driving bearing piles close, after the manner of sheeting piles, at per lineal foot			
7. Fitting and fixing wale-pieces and sills, at per lineal foot			
8. Iron-work in screw bolts, spikes, and straps, at per ℔.			
9. Clay puddle in coffer-dams, ramming behind walls, and for roadway, at per cubic yard			
10. Upfilling with broken Whinstone between the heads of bearing piles, at per cubic yard			
11. Coursed pitching of cut freestone laid between the sills and foundations, at per cubic foot			
12. Masonry in face-work and hearting of the abutments			

£. s. d.

and piers under the level of summer water mark, at per cubic foot

13. Masonry in face-work and hearting of the wing walls, at per cubic foot
14. Masonry in face-work of abutments and piers from the level of summer water mark to the top of the impost course, at per cubic foot
15. Masonry in hearting of abutments and piers from the level of summer water mark to the top of the impost course, at per cubic foot
16. Masonry in springing courses of abutments and piers, at per cubic foot
17. Masonry in arches, at per cubic foot
18. Broached and chamfered Aisler work, at per superficial foot
19. Coursed rubbled masonry, at per cubic foot
20. Good common hammer-dressed rubble masonry, at per cubic foot
21. Pilasters built in courses, at per cubic foot
22. Frieze over pilasters, at per cubic foot
23. Cornice of bridge, at per lineal foot
24. Parapet walls of bridge, at per lineal foot
25. Curb-stones for foot pavement, at per lineal foot
26. Sky-drains for roadway, at per lineal foot
27. Foot pavement, at per superficial yard
28. Embanking approaches, stuff removed not exceeding 300 yards, at per cubic yard
29. Metalled roadway, at per cubic yard
30. Retaining walls, at per do.
31. Retaining walls with coursed face-work and rubble backing, at per cubic yard
32. Removal of excavated stuff per hundred yards, exceeding 300 yards, and not exceeding 600 yards, at per cubic yard
33. Hammer-dressed cope for spandril walls, at per lineal foot
34. Hammer-dressed pavement, at per superficial yard
35. Centre frames of arches, and work connected therewith, at per cubic foot
36. Built close drains measuring 2 feet 6 inches square, at per cubic foot

£. s. d.

37. If the width of the bridge shall be increased, and the roadway made broader than is described or shown in the plan and foregoing specification, the whole charge for extra work, including labour, materials, and implements of all kinds rendered necessary on this account for the work, shall not for every additional lineal foot of breadth upon the whole length of the bridge by which the roadway and approaches are so increased exceed the rate or sum of

38. And so in like manner and proportion for any diminution of breadth on the whole length of the roadway and approaches of the bridge, from that which is described or shown in the plan and foregoing specification, a rateable and corresponding deduction shall be made per foot of breadth, at the rate of

39. If the foundation of either of the abutments of the bridge, measuring from the point towards the river landward to the points or springing of the horizontal arch, shall be sunk or carried to a greater depth than is described or shown in the plan or foregoing specification, the whole charge for extra work, including labour, materials, and implements of all kinds rendered necessary for the work on that account, shall not for every additional vertical foot by which the depth is increased exceed the rate or sum of

40. And so in like manner and proportion for any diminution of depth, as aforesaid, by which the foundation of either of the abutments of the bridge is lessened in the specified depth, a rateable and corresponding deduction shall be made per foot perpendicular at the rate of

41. If the foundation of any of the piers of the bridge shall be sunk or lowered to a greater depth than is described and shown in the plan and foregoing specification, the whole charge for extra work, including labour, materials, and implements of all kinds rendered necessary for the work, on this account, shall not for every additional perpendicular foot by which the depth is increased exceed the rate or sum of

42. And so in like manner and proportion for any diminution of depth, as aforesaid, by which the foundations

£. s. d.

of any of the piers of the bridge are lessened in the specified depth, a rateable deduction shall be made per vertical foot at the rate of

43. If instead of the coupled pilasters, with their basements, entablatures, and connecting works, the retaining walls, cornice, and parapet walls be continued and extended throughout the elevation of the bridge in a plain form, such alteration will form a deduction upon the contract price, amounting to the sum of

Note.—All charges for extra work specified in the above schedule for any operation or article referable to labour or materials necessary for protecting works of this kind, though not enumerated and expressed, are understood to be provided for and covered in the charges respectively specified in the schedule, or otherwise left to the determination of the engineer.

(Signed and dated by the Offerer.)

SUPPLEMENT

TO VOL. I.

THE MATHEMATICAL PRINCIPLES

OF

MR. DREDGE'S SUSPENSION BRIDGE.

THE MATHEMATICAL PRINCIPLES

OF

MR. DREDGE'S SUSPENSION BRIDGE.

A NOVEL and elegant principle has lately been adopted in the construction of *Suspension Bridges*,—a principle which promises to be of the greatest utility and importance, both as regards the quantity of material employed, and the stability and strength which it confers upon the structure.

One of the distinguishing features of this very ingenious invention, which is due to Mr. DREDGE, of Bath, consists in making the chains of sufficient magnitude and strength at the points of suspension, to support with safety the greatest permanent and contingent load, to which, under the circumstances of locality, they are ever likely to be exposed; and from thence, to taper or diminish them gradually to the middle of the bridge, where the strain becomes essentially evanescent.

The gradual diminution of the chains, however, is not the only circumstance which characterizes this mode of construction, and marks its superiority over the methods employed by other engineers. The suspending rods or bars that support the platform or road-way, instead of being hung vertically or

at right angles to the plane of the horizon, are inclined to it in angles which vary in magnitude from the abutments to the middle of the bridge, where the obliquity, as well as the stress upon the chains, attains its minimum value.

The principle developed by this obliquity of the suspending rods is singularly beautiful; but much judicious management is necessary on the part of the engineer, to fix upon that degree of obliquity which shall produce the greatest effect. Each bar is considered to perform its part in supporting the load, in proportion to its distance from the abutment, drawn into the sine of the angle of its direction, so that the entire series of suspending bars, transmits the same tension to the points of support as would be transmitted by a single bar reaching from thence to the middle of the bridge.

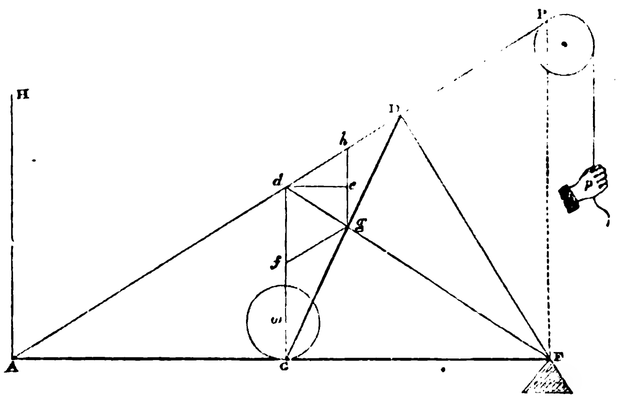
If this individual bar be considered as a straight inflexible line, the principles of calculation are identical with those of a lever when sustained in equilibrio by a single force applied at the remote extremity, and acting in the direction of the sustaining bar. But when the bar is curved or polygonal, as it necessarily must be in the case of a chain employed in the actual construction of a bridge, the process becomes a little more complicated, since it requires all the componant forces to be referred to the several portions of the chain as resultants; we mean such portions of it as are comprehended between any two contiguous bars. In every other respect, the principles which regulate the calculations are the same; and for this reason, it will be sufficient to establish the theory in reference to the straight line only, since the reader who is familiar with the *Composition* and *Resolution* of *Forces*, will find no difficulty in extending the process to a curve, and he who is not, will derive but little benefit from a perusal of the subject which is now to be laid before him.

Nothing can be more simple than the doctrine of the lever,

when sustained in equilibrio by a single force applied at a given point, and acting in a given direction: this is explained in every treatise on the principles of mechanical science, and is consequently within the reach of every individual connected with that or kindred pursuits: but a more important case of the problem remains to be considered, viz., that in which the equilibrium is maintained by a system of forces applied at different points in the length of the lever, and acting in different directions with respect to the horizon. This is the case that becomes assimilated to a suspension bridge, when the chains and suspending rods are simultaneously called into action in such a manner, that their joint effects shall be competent to maintain the system in a state of balanced rest.

In the case of a single force applied at the remote extremity of the lever, and having its direction inclined to the horizon in a given angle, while the load is applied at the middle of the length, and acting in the direction of gravity. Let A F (fig. 1)

Fig. 1.



be a lever, having one extremity resting on the fulcrum at F, and conceive it to be sustained in equilibrio by a power or force *p* applied at A the other extremity, and acting in the direction A P.

Conceive the lever, thus circumstanced, to be a heavy and

perfectly inflexible bar, of uniform figure and density in all its parts, and suppose the whole of its weight to be accumulated in G the centre of gravity; it is then reduced to the case of a straight inflexible lever void of weight, and having a load of given magnitude applied at its middle point.

From the fulcrum F draw the straight line F D perpendicular to A P, the direction of the sustaining force; then, by the principles of plane trigonometry, we have the following proportion, viz.,

$$\text{rad.} : \sin. \angle \text{FAD} :: \text{FA} : \text{FD}.$$

Or, by reducing the analogy and making radius = 1, it is

$$\text{FD} = \text{FA} . \sin. \angle \text{FAD}.$$

Now, F D is the perpendicular distance between the fulcrum and the direction in which the force acts; consequently, by the property of the lever,

> *The power acting in the direction* A P, *is to the weight acting in the direction of gravity, as the distance* F G *is to the distance* F D. *That is, the power and the weight are reciprocally as the distances* F G *and* F D.

Let p = the magnitude of the force which is applied at A, and acting in the direction A P,

d = A F, the whole length of the lever, or the distance from the fulcrum of the point at which the force acts,

ϕ = F A P, the angle of direction, or that contained between the plane of the horizon and A P the direction of the force,

w = the weight of the lever or platform, supposed to be concentrated at G the middle of its length,

$\delta = \frac{1}{2}d$ = F G, the distance between the fulcrum and the point where the weight acts.

Then, by the property of the lever just enunciated, the state of

equilibrium in terms of our notation is expressed by the following analogy, viz.,

$$p : w :: \delta : d_n \sin. \phi_n .$$

Therefore, by multiplying extremes and means, the *equation of condition* is

$$p\, d_n \sin. \phi_n = \delta\, w. \qquad \text{(A)}$$

And by division, the value of the suspending power becomes

$$p = \frac{\delta}{d_n} . w \operatorname{cosec}. \phi_n . \qquad \text{(B)}$$

Now, by the principles of mechanics, the effect must obviously be the same at whatever point in the line A P the force may be applied; so that, if A P be a cord or chain supporting the weight w by its tension, the measure of that tension is the same at every point, provided that its inclination in respect of the horizon is constantly the same. And if P be a point vertically situated with respect to the fulcrum F, let the cord or chain A P be suspended at P, in such a manner as to be kept perfectly tight by the strain which is excited in maintaining the equilibrium; then the value of the strain upon the point of suspension at P is expressed by equation (B).

Reverting to the equation of condition, and substituting $\frac{1}{2}d_n$ for δ, the expression for the state of equilibrium becomes

$$p \sin. \phi_n = \tfrac{1}{2}\, w, \qquad \text{(C)}$$

or, by division, we obtain for the equilibrating power,

$$p = \tfrac{1}{2}\, w \operatorname{cosec}. \phi_n . \qquad \text{(D)}$$

This expression for the measure of the power which maintains the system in a state of rest is exceedingly simple; but being limited to the case where the weight acts at the middle of the lever, and the sustaining power at the remote extremity, it will be more advantageous in our subsequent inquiries to retain

the general value, as expressed above, in terms of the respective distances d_n and δ; because, in as far as the lever is concerned, the symbol δ may partake of all values less than d_n, the entire length of the bar on which the forces act.

If the power p, instead of acting in the direction A P oblique to the horizon, should act in the direction A H perpendicular to it, then $< \mathrm{FAH} = \phi_n = 90°$, and cosec. $\phi_n = 1$; in which case we have

$$p = \tfrac{1}{2} w. \qquad \text{(E)}$$

This is manifest; for one half the weight is sustained by the fulcrum, and the other half by the power applied at A.

In what precedes, we have established the equation of condition by reference to the principles of the lever only; but the same thing can readily be accomplished by the *Resolution* of *Forces*, and by the same means we at once obtain the magnitude of the oblique strain upon the fulcrum or abutment, and also the horizontal thrust. Through the point G (fig. 1) draw the straight line G d at right angles to A F, and meeting A P the direction of the power in d. Draw d F and D G, and from the point d on the vertical line d G set off df at pleasure, to denote the magnitude of the weight w acting in the direction of gravity. Upon df complete the parallelogram $dfgh$, and fg or dh will represent the magnitude of the force in the direction A P, while the diagonal dg denotes the oblique pressure on the fulcrum, or the magnitude of the force in the direction d F.

Since the angles A D F and A G d are right angles by construction, the quadrilateral figure F D d G can be inscribed in a circle, and because the angles F D G and F d G are subtended by the same chord F G, they are equal between themselves; the angles D F G and dfg are also equal, each of them being equal to the complement of the given angle P A F. Con-

sequently, the triangles F D G and fdg are similar, and their homologous or corresponding sides proportional; so that

$$FD : fd :: FG : fg.$$

Now F D, as we have already seen, is equal to F A . sin. $< FAD = d_n \sin. \phi_n$, and $fd = w$, F G and fg being respectively represented by the symbols δ and p; hence we get

$$d_n \sin. \phi_n : w :: \delta : p,$$

from which, by equating the products of the extremes and means, we obtain

$$p\, d_n \sin. \phi_n = \delta\, w. \text{ (the same as equation A).}$$

In the plane triangle dfg we have given the two sides df and fg, with the contained angle dfg, to find the third side dg; and for this purpose, a well known theorem in plane trigonometry gives

$$dg = \left\{ w^2 + p^2 - 2\, wp \sin. \phi_n \right\}^{\frac{1}{2}} \qquad \text{(F)}$$

But, according to equation (B), the value of p in terms of d_n, δ, w and ϕ_n is $\frac{\delta}{d_n} . w$ cosec. ϕ_n; and consequently, its square is $\frac{\delta^2}{d_n^2} . w^2$ cosec.^{2} ϕ_n; therefore, by substituting these values for p and p^2 within the brackets, we get

$$dg = w \sqrt{1 + \frac{\delta^2}{d_n^2} \text{cosec.}^2 \phi_n - \frac{2\,\delta}{d_n} \text{cosec.}\, \phi_n \sin.\, \phi_n}.$$

By the principles of trigonometry, the cosecant of any arc or angle to radius unity, is the same as the reciprocal of the sine to the same radius; hence we have cosec. ϕ_n sin. $\phi_n = 1$, so that the expression for the oblique thrust becomes

$$r = \frac{w}{d_n} \sqrt{d_n^2 - 2\, d_n \delta + \delta^2 \text{cosec.}^2 \phi_n}. \qquad \text{(G)}$$

Where the symbol r denotes the resultant of the weight or load w and the oblique force p, and corresponds in magnitude and direction to the oblique thrust on the fulcrum at F.

There is still another quantity remaining to be determined, the magnitude of which may sometimes be required in a case of actual construction; we mean the horizontal thrust upon the abutment, occasioned by the joint effects of the weight and the oblique power, when exerting themselves with full intensity in their respective directions.

Let dg (fig. 1), which represents the oblique thrust upon the abutment, be resolved into the two forces de and ge, the one of them being parallel to the horizon, and the other perpendicular to it; then it is obvious that in so far as the thrust upon the abutment is concerned, the effect of the vertical force is wholly lost, that of the horizontal force only being employed in producing the strain.

Since, by construction, the straight line de is parallel to A F, the angle hde is equal to the angle $\text{P A F} = \phi_n$, and we have already seen, equation (B), that fg or its equal dh is expressed by $\frac{\delta}{d_n} . w \text{ cosec. } \phi_n$; therefore by the principles of plane trigonometry we get

$$\text{rad.} : \frac{\delta}{d_n} . w \text{ cosec. } \phi_n :: \cos. \phi_n : de.$$

Therefore, if h be put to denote the horizontal thrust upon the abutment, the reduction of the above proportion will give

$$h = \frac{\delta}{d_n} . w \text{ cosec. } \phi_n \cos. \phi_n .$$

But, according to the calculus of sines, the product of the cosine and cosecant of any arc or angle is equal to the cotangent; hence we have $\text{cosec. } \phi_n \cos. \phi_n = \cot. \phi_n$, from which, by substitution, we get

$$h = \frac{\delta}{d_n} . w \cot . \phi_n . \qquad \text{(H)}$$

It therefore appears that the equations (B), (G), and (H), exhibit the several particulars required in and arising from the equilibration of a lever; and in order that these particulars may be clearly understood, we shall now proceed to show in what manner the equations are to be reduced, by applying them to the resolution of a numerical example.

Example.—Suppose the length of a lever to be 56 feet, each foot in length being equal to a weight of 40·56 lbs. avoirdupoise; now, if one extremity of this lever rests upon a fulcrum, while it is sustained in equilibrio by a force applied at the other extremity, and acting in a direction which makes an angle of 32 degrees with the horizon; what is the magnitude of the sustaining force, and what are the thrusts excited upon the fulcrum, both in reference to the oblique and horizontal direction?

The first demand of this question is fulfilled by equation (B), the second by (G), and the third by (H); in all of which, the value of δ is 28 feet, being equal to one half the length of the lever. And since the system is placed under the same conditions as it would be, if a load equivalent to the weight of the lever were applied at its centre of gravity, the lever itself being then considered as without weight, it follows that

$$w = 56 \times 40{\cdot}56 = 2271{\cdot}36 \text{ lbs.}$$

Since the angle of direction of the suspending power is 32 degrees, its natural cosecant is 1·88708, and the numerical expression for the magnitude of the power by which the equilibrium is maintained is

$$p = \frac{28 \times 2271{\cdot}36 \times 1{\cdot}88708}{56} = 2143{\cdot}12 \text{ lbs.}$$

The logarithmic process for the reduction of this expression is as follows, viz.,

$\delta = 28$ feet	log.	1·4471580	add.
$w = 2271{\cdot}36$ lbs.	log.	3·3562859	
$\phi_n = 32$ degrees	log. cosec.	0·2757903	
$d_n = 56$ feet	ar. co. log.	8·2518120	

Therefore the value of p, the equilibrating power,
is 2143·12 lbs. . . . . log. 3·3310462 sum.

Hence it appears, that a power of 2143·12 lbs., acting at the remote extremity of the lever, and having its direction inclined to the horizon in an angle of 32 degrees, will balance a load of 2271·36 lbs. applied at the middle of the length, and acting in the direction of gravity, the lever itself having no weight.

The data remaining as above, the numerical expression for the value of the oblique thrust on the abutment is

$$r = \frac{2271{\cdot}36}{56}\left\{56^2 - 2 \times 56 \times 28 + 28^2 \times 1{\cdot}88708^2\right\}^{\frac{1}{2}}$$

By a slight examination of the quantities within the brackets, it will readily appear that the second or negative term is precisely equal to that which precedes it; consequently, they destroy one another, and the expression reduces to the following, viz.,

$$r = \frac{2271{\cdot}36 \times 28 \times 1{\cdot}88708}{56} = 2143{\cdot}12 \text{ lbs.}$$

From this we infer that when the load is applied at the middle of the lever, and the balancing power at the remote extremity, the magnitude of the oblique thrust on the fulcrum is precisely the same as the magnitude of the sustaining power. This is also manifest from the figure; for the triangle $g\,d\,h$ or $d\,g\,f$ is isosceles.

Finally, the data still remaining, the numerical expression for the magnitude of the horizontal thrust on the fulcrum becomes

$$h = \frac{28 \times 2271{\cdot}36 \times 1{\cdot}6003345}{56} = 1817{\cdot}47 \text{ lbs.}$$

Where 1·6003345 is the natural cotangent of 32 degrees; hence the following logarithmic process,

$\frac{\delta}{d} = \frac{28}{56}$ . . . . log. 9·6989700

$w = 2271·36$ . . . . log. 3·3562859

$\phi = 32°$ . . . . log. cot. 0·2042108

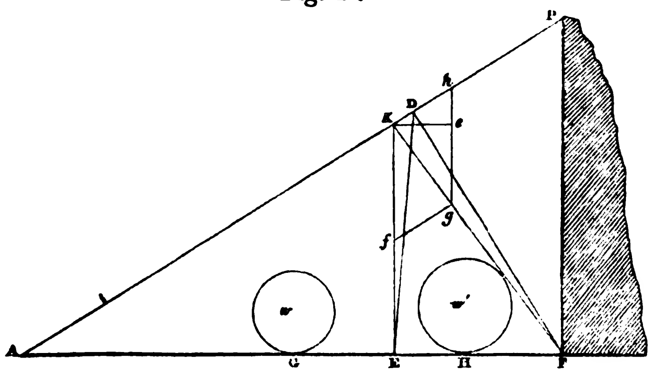
Therefore the value of h, the horizontal thrust, is

1817·47 lbs. . . . . log. 3·2594667 sum.

The preceding investigation has reference only to the weight of the lever or platform, which is constantly the same; but in the case of a bridge, this constancy of weight cannot obtain by reason of the transit loads to which it is exposed; and as the loads of transit are continually varying their distances from the fulcrum, and thereby producing different effects at every instant, it becomes necessary, in order to render our investigation complete, to show in what manner these effects are to be estimated, and incorporated with the effect produced by the weight of the platform.

Let w (fig. 1[a]), denote the weight equivalent to that of the

Fig. 1[a].



platform, applied at G the centre of gravity or middle point of the lever, and let w' be any other load applied at the point H.

Then it is manifest, from the principles of the lever, that the two weights w and w' will, according to their situations, perform the same office in equipoising the system, as would be performed by a single weight equivalent to their sum applied at E their common centre of gravity, and acting in a direction perpendicular to the horizon. Divide the distance G H into two parts G E and H E, which are to one another reciprocally as the weights applied at G and H; then is E the position of the load, which is equivalent to $w + w'$. Through the point E, and perpendicular to F A, draw the straight line E K, meeting A P in K; and let F D be perpendicular to A P. Draw D E and F K, and upon K E set off K f, equal to the compound load $w + w'$, and complete the parallelogram K $f\,g\,h$; then will $f\,g$ or K h denote the force in direction A P, and K g the oblique pressure on the fulcrum at F, arising from the conjoint action of the constant weight w and the extraneous weight w'. Through the point K draw K e at right angles to $h\,g$, and K e will denote the horizontal thrust upon the fulcrum, produced by the conjoint action of the two forces already specified; and if the load w' should move forward or backward to any other point of the lever, the same mode of construction will still apply, so that the principle is general, whether the load be constant or variable.

If the symbol δ denote the distance F E between the fulcrum F and the common centre of gravity of the two weights w and w', the other quantities remaining as before, then the equation of *condition* for the constant and variable load becomes

$$p\, d_n \sin.\, \phi_n = \delta\, (w + w'), \qquad (\mathrm{H}^a)$$

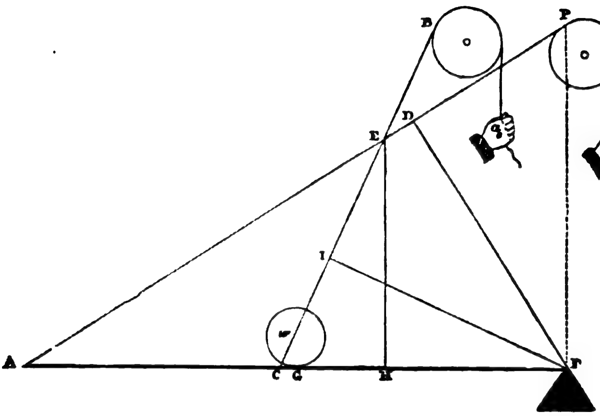
and the absolute value of the force in direction A P is

$$p = \frac{\delta}{d_n} \cdot (w + w')\, \text{cosec.}\, \phi_n. \qquad (\mathrm{H}^b)$$

In the next place, instead of the lever being sustained in

equilibrio by a single power p, applied at A and acting in the direction A P (fig. 2), let us suppose that the equilibrium is

Fig. 2.



produced by the joint effects of two forces b_o and a_o, the one being applied at A the extremity of the lever, and acting in the direction A P, while the other is applied at some intermediate point of the lever, as C, and acting in the direction C B, the angles of inclination P A F and B C F being respectively denoted by the symbols ϕ_n and ϕ_o, while the distances A F and C F are indicated by d_n and d_o, the symbol δ being the same as in the preceding case; namely, the distance between the fulcrum and the point where the weight acts; that is, the distance G F.

Let the point C, at which the subsidiary force a_o is applied, lie between the points A and G; that is, between the middle of the lever and its extreme point at A; then, since the magnitude of the weight w and its distance from the fulcrum are the same as before, its effect upon the lever will manifestly be the same also, and consequently the joint effects of the two forces b_o and a_o, which produce the equilibrium in this case, will

be equivalent to the effect of the single force p, by which it was previously produced.

But by the laws of mechanics the effect of any force which acts upon a lever at a given distance from the fulcrum, and in a given direction,—

> *Is expressed by the magnitude of the given force, drawn into the distance between the fulcrum and the point at which it acts, and again into the sine of the angle of its direction.*

We have already seen, equation (A), that the effect of a single force which balances the weight of the lever is expressed by $p\,d_n \sin. \phi_n$; this, therefore, is the quantity to which the effects of the forces b_0 and a_0, now balancing the weight, must be equal, and with which they must jointly be compared.

Now, by the property of the lever as above enunciated, the effect of the force b_0 applied at A, and acting in the direction A P, is $b_0\,d_n \sin. \phi_n$, the distance A F and angle P A F being the same as before. And in like manner the effect of the force a_0 applied at the point C, and acting in the direction C B, is $a_0\,d_0 \sin. \phi_0$; consequently, by taking the sum of these effects, the equation of condition or equilibrium in the case of two sustaining forces becomes

$$a_0\,d_0 \sin. \phi_0 + b_0\,d_n \sin. \phi_n = p\,d_n \sin. \phi_n.$$

But by equation (A) we have $p\,d_n \sin. \phi_n = \delta\,w$; consequently, by substitution, we obtain

$$a_0\,d_0 \sin. \phi_0 + b_0\,d_n \sin. \phi_n = \delta\,w. \qquad \text{(I)}$$

The composition of this equation is very simple; for from the fulcrum F draw the straight lines F D and F I respectively perpendicular to A P and C B, the direction in which the forces

act. Then, since $AF = d_n$ and $CF = d_0$, while the angles PAF and BCF are respectively denoted by the symbols ϕ_n and ϕ_0, it follows from the principles of plane trigonometry, that

$$FD = d_n \sin. \phi_n, \quad FI = d_0 \sin. \phi_0.$$

And multiplying these distances by the magnitudes of the respective powers, the products will indicate the mechanical effects, and the sum of those effects must be equal to the magnitude of the weight w drawn into the distance FG; and this equality constitutes the equation of *condition* or *equilibrium* given above.

If we transpose the first term of equation (I), the effect of the force b_0 becomes

$$b_0 d_n \sin. \phi_n = \delta w - a_0 d_0 \sin. \phi_0,$$

and by division we obtain

$$b_0 = \frac{\delta w - a_0 d_0 \sin. \phi_0,}{d_n \sin. \phi_n}.$$

But, according to the calculus of sines, the sine and cosecant of any arc or angle to radius unity are reciprocally equal; consequently, a more elegant expression for the magnitude of b_0 is that which follows, viz.,

$$b_0 = \frac{\left(\delta w - a_0 d_0 \sin. \phi_0\right) \operatorname{cosec.} \phi_n}{d_n}. \qquad \text{(K)}$$

This supposes the magnitude of the force a_0, as well as all the other quantities composing the equation except b_0, to be known; but if b_0 be the given force, the equation expressing the value of a_0 becomes

$$a_0 = \frac{\left(\delta w - b_0 d_n \sin. \phi_n\right) \operatorname{cosec.} \phi_0}{d_0}. \qquad \text{(L)}$$

If the equations (B) and (K) be compared with each other, it will readily appear that the former exceeds the latter by the quantity $\frac{a_0 d_0 \sin.\phi_0}{d_n} \times \operatorname{cosec.}\phi_n$; for it is obvious that

$$p-b_0=\frac{\delta}{d_n}.w\operatorname{cosec.}\phi_n-\frac{\left(\delta w-a_0 d_0\sin.\phi_0\right)\operatorname{cosec.}\phi_n}{d_n}=\frac{a_0 d_0\sin.\phi_0\operatorname{cosec.}\phi_n}{d_n}.$$

We therefore infer, that in the case of a single subsidiary force a_0, as in fig. 2, the tension upon the cord or chain A P is less than the tension in fig. 1, by the quantity

$$\frac{a_0 d_0\sin.\phi_0\operatorname{cosec.}\phi_n}{d_n},$$

so that the diameter or section of the cord or chain may be diminished in the same proportion, the force in the direction C B diminishing the strain to that extent.

If the force a_0, which is applied at the point C, have its direction perpendicular to the length of the lever A F; then $\sin.\phi_0 = 1$, and the effect in that case is simply $a_0 d_0$ instead of $a_0 d_0 \sin.\phi_0$, and the corresponding value of b_0 is therefore

$$b_0=\frac{\left(\delta w-a_0 d_0\right)\operatorname{cosec.}\phi_n}{d_n}. \qquad \text{(M)}$$

If the equations (B) and (M) be now compared with each other, it will be seen that the former exceeds the latter by the quantity $\frac{a_0 d_0 \operatorname{cosec.}\phi_n}{d_n}$; for, by subtraction, it is

$$p-b_0=\frac{\delta}{d_n}.w\operatorname{cosec.}\phi_n-\frac{\left(\delta w-a_0 d_0\right)\operatorname{cosec.}\phi_n}{d_n}=\frac{a_0 d_0\operatorname{cosec.}\phi_n}{d_n}.$$

Now, when the angle of inclination ϕ_0 is of any magnitude

whatever less than 90 degrees or a right angle, sin. ϕ_0 is less than unity, and, consequently, this last remainder is greater than the former by the quantity $\frac{a_0 d_0 \text{ cosec. } \phi_n}{d_n}\left(1 - \text{sin. } \phi_0\right)$; for by subtraction we get

$$\frac{a_0 d_0 \text{ cosec. } \phi_n}{d_n} - \frac{a_0 d_0 \text{ sin. } \phi_0 \text{ cosec. } \phi_n}{d_n} = \frac{a_0 d_0 \text{ cosec. } \phi_n}{d_n}\left(1 - \text{sin. } \phi_0\right).$$

From this we infer that when the direction of the force a_0 is perpendicular to the lever, the magnitude of the force b_0 in direction A P necessary to maintain the equilibrium, is less than when the direction of the force a_0 is oblique. This inference, at first sight, would seem to be at variance with the principles insisted on by Mr. Dredge; and it manifestly is so, if we conceive the vertical and oblique forces to be applied at the same point of the lever; but if we suppose them to be referred to the same point of the cord or chain A P, the case will be very different; for then the quantity indicated by the symbol d_0 diminishes, and the quantity of diminution is proportional to the cosine of the inclination or obliquity. For, draw the perpendicular E H, then C F is the length of the lever in the case of the oblique force in direction C E, and H F its length for the vertical force in direction H E; but C H = C F − H F, and C H = C E cos. ϕ_0; for, by the principles of plane trigonometry, it is

$$\text{rad.} : \text{C E} : : \text{cos. } \phi_0 : \text{C H} = \text{C E cos. } \phi_0.$$

In this equation, however, the quantity C E is still unknown, but it may be determined in the following manner:

$$\text{A C} = \text{A F} - \text{C F} = d_n - d_0,$$

and by the property of the plane triangle, as demonstrated in the 32nd proposition of the first book of Euclid's Elements of Geometry, the angle E C F is equal to the sum of the angles E A C and A E C; therefore by transposition it is

$$A\,E\,C = \phi_0 - \phi_n,$$

and by trigonometry we have

$$\sin.\left(\phi_0 - \phi_n\right) : d_n - d_0 :: \sin.\,\phi_n : C\,E,$$

and by reducing the analogy it becomes

$$C\,E = \frac{\left(d_n - d_0\right)\sin.\,\phi_n}{\sin.\left(\phi_0 - \phi_n\right)}. \tag{N}$$

Let this value of C E be substituted instead of it in the above value of C H, and we obtain

$$C\,H = \frac{\left(d_n - d_0\right)\sin.\,\phi_n\cos.\,\phi_0}{\sin.\left(\phi_0 - \phi_n\right)}. \tag{O}$$

By a well known theorem in the calculus of sines, we have

$$\sin.\left(\phi_0 - \phi_n\right) = \sin.\,\phi_0\cos.\,\phi_n - \cos.\,\phi_0\sin.\,\phi_n;$$

therefore, by substitution, we get

$$C\,H = \frac{\left(d_n - d_0\right)\sin.\,\phi_n\cos.\,\phi_0}{\sin.\,\phi_0\cos.\,\phi_n - \cos.\,\phi_0\sin.\,\phi_n},$$

and by expunging the factor $\sin.\,\phi_n\cos.\,\phi_0$ from both terms of the fraction, it finally becomes

$$C\,H = \frac{d_n - d_0}{\tan.\,\phi_0\cot.\,\phi_n - 1}. \tag{P}$$

If from C F the leverage for the oblique force in direction C B, we subtract the value of C H, as found above, the remainder H F will be the leverage for the vertical force in direction H E. Thus we have

$$\text{H F} = \frac{d_0 \tan.\phi_0 \cot.\phi_n - d_n}{\tan.\phi_0 \cot.\phi_n - 1}. \qquad \text{(Q)}$$

When the value of the distance H F, as found above, is less, equal to, or greater than F I $= d_0 \sin.\phi_0$, the effect of the vertical force will be less, equal to, or greater than that of the oblique force; by this we mean, that the magnitude of the force being the same in both cases, its effect in the vertical direction will be less, equal to, or greater than its effect in the oblique direction, according as H F is less, equal to, or greater than F I.

Since E is a fixed point through which the directions of both the vertical and oblique forces must pass, it follows that while the distance F H is constant, the distance F C may partake of all magnitudes between F A and F H, while the angle E C F may librate between the angles F H E and F A P; that is, it may vary from a right angle to ϕ_n, which is its minimum limit.

From this view of the subject it is obvious that the perpendicular F I depends upon the magnitude of the angle E C F; and because the point C may move either towards A or H, while the straight line C B passes constantly through E, it follows, that the inclination admits of such a magnitude as to give the force in the direction C B a maximum effect; that is, when $d_0 \sin.\phi_0$ is a maximum or the greatest possible.

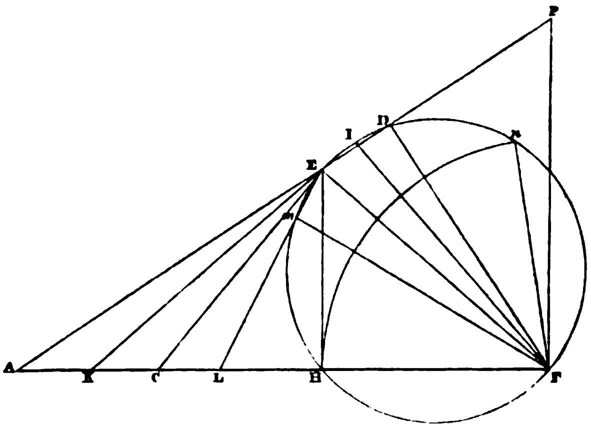
The method of determining the angle which gives to the oblique force its maximum effect is very simple; but if such a principle were adopted in the construction of suspension bridges, the results in many instances would differ very widely from the practice of the inventor, and especially as applied to the suspending rods in the *Victoria Bridge* across the Avon at Bath.

From drawings which we have seen of that structure, it would appear that the angles of direction diminish according to some law, as we recede from the abutments towards the centre

of the bridge; but in making the obliquity such as to give the force in any direction its maximum effect, the angles continually increase instead of decreasing, as will become manifest from the following constructions.

Let A F (fig. 3) be a lever resting on the fulcrum at F, and

Fig. 3.



sustained in equilibrio by a cord or chain applied at A, and fastened to a hook or some other immoveable object at P.

Let E be any point in the length of the cord, at which a subsidiary force is to be applied in order to lessen the strain upon A E, the lower portion thereof. The point C is required, so that C E being joined, a given force acting in the direction C E may produce a maximum effect.

Draw the straight line F E to connect the fulcrum with the given point, and through the point E, draw E C at right angles to F E, and meeting the lever A F in C; then is C E the direction of the force required. Upon F E, as a diameter, describe the circle F H E, intersecting the lever in the point H, and draw E H; then is E H perpendicular to the lever in H, so that H E would be the direction of the vertical force passing through E the given point.

If the point C be supposed to move towards A in such a

manner that C E may become K E; then it is manifest that K E produced will intersect the circumference of the circle again in I; draw F I, and F I will be perpendicular to K E, the direction of the given force; this is obvious, for the angle in a semicircle is a right angle. Now F I is manifestly less than F E, for F E is the diameter of the circle F H E, and by the property of the circle, the diameter is the greatest chord that can be inscribed in it. When the point K coincides with A, the point I coincides with D, in which case $F D = d_n \sin. \phi_n$.

Again, suppose the point C to move towards H until it arrives at the point L; then L E being drawn will necessarily intersect the circumference of the circle in the point *m*. Draw F *m*, which, by the property of the circle, will be perpendicular to L E, the direction of the given force supposed to be applied at L. Now, since F E is the diameter of the circle F H E, and F *m* any chord in it not passing through the centre, F E is greater than F *m*; but it has also been shown to be greater than F I; and since the effect of the force in the directions K E, C E, and L E, is represented by the perpendiculars F I, F E, and F *m*, it follows, that the effect in the direction C E is a maximum; hence the truth of the construction is manifest. When the point L coincides with H, the straight line L E coincides with H E, and H E becomes the direction of the vertical force, the effect of which is represented by F H.

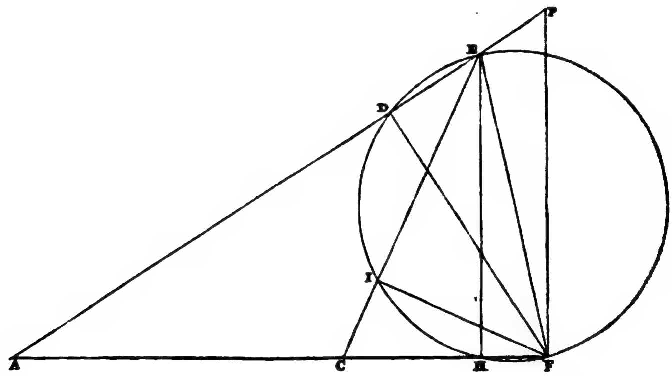
Since H and D are the points in which the circle is intersected by the lever A F, and the cord or chain A P, it follows, that the effect of the given force may be represented by all the chords that can be drawn in the circle between the points D and H, F D being the minimum value when the point C moves towards A, and F H the minimum value when it moves in the opposite direction. And all these values will manifestly be greater than F H; for from the centre F with the distance F H, describe the arc H *n* intersecting the circumference of the

circle in n; then it is evident that all the chords originating at F, and meeting the circumference between the points n and H, must be greater than F n or F H. Thus far, therefore, there is a manifest advantage in causing the force to act in the oblique direction.

The preceding construction determines the point in the lever at which the oblique force must be applied to produce a maximum effect for all points in the cord or chain between A and D; but it does not apply to points on the opposite side of D towards P, for in that case the direction of the force would meet the lever on the opposite side of A, which is manifestly contrary to the conditions of the problem as applied to the construction of bridges; for all the suspending bars on one side are supposed to meet the platform between the abutment and the middle of the bridge. The fact is, the direction which gives the maximum value is the direction of the cord itself, the force being applied at the point A; this will be obvious from the subjoined construction.

Let A F (fig. 4) be the lever resting upon the fulcrum at F,

Fig. 4.



and A P the direction of the force, which, acting singly at A, maintains the equilibrium. Let C be the point at which the subsidiary force is applied, and C E the direction in which it

acts; then, *it is proposed to show, that the greatest effect will be produced by the given power when it acts in the direction* A E *coincident with the cord or chain* A P.

Draw the straight line F E, connecting the fulcrum with that point in the cord through which the direction of the force passes, and upon F E as a diameter describe the circle F H D E, intersecting the lever A F in the points F and H; the cord A P in the points D and E, and C E the direction of the force in the point I. Draw F I, F D, and E H, these will respectively be perpendicular to the lines C E, A P, and A F. Now, it is obvious, that if the point C moves along the lever A F, either towards A or H, the straight line C E will intersect the circle in points towards D or H accordingly, and the perpendicular F I will increase or diminish according as the intersections fall towards D or H.

When the point C coincides with A the extremity of the lever, the straight line C E will coincide with A E, the direction of the cord, and the perpendicular F D will then attain its maximum value; hence the truth of the proposition is manifest.

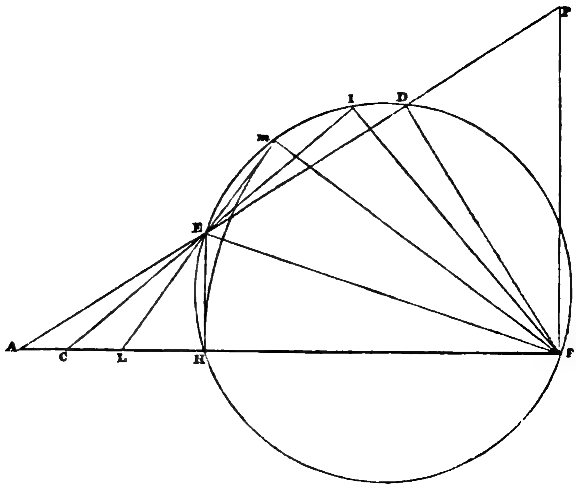
Again, when the point C coincides with the point H, the straight line C E will coincide with the perpendicular H E; and in that case the value of F H is a minimum; from which we infer that in all cases where the direction of the subsidiary force intersects the cord between the points D and P, the effect is the least possible when the force is exerted in a vertical direction.

In both the preceding constructions the position of the point E has been so assumed as to give the force, when acting in an oblique direction, a greater effect than when it acts in a vertical direction; and lest the inattentive reader should imagine that this is universally the case, we shall, in what immediately follows, endeavour to show that the effect of the

force acting vertically, may also be either equal to or greater than the effect of the same force acting obliquely, when the directions in both cases pass through the same point of the cord.

Let A F (fig. 5) be the lever, similarly situated as in all

Fig. 5.



the preceding cases, A P being the direction of the single equilibrating power. Take any point E in A P, and let the point E thus assumed be that through which the direction of the subsidiary force has to pass.

Draw F E, and upon F E as a diameter describe the circle F H E D, intersecting the lever in F and H, and the cord or chain A P in E and D. Let C be any point in the lever at which the subsidiary force is applied, and through the point E draw the straight line C E, meeting the circumference of the circle in I; then is C E the direction of the force, and F I perpendicular to C I, the representative of its effect. About the fulcrum F as a centre, with the distance F H describe the arc H *m* intersecting the circumference of the circle in *m*, and draw the straight lines F *m* and *m* E L; then is L E the direction of the oblique force acting at the point L, and the perpendicular

F *m* is the representative of its effect. But F *m* is equal to F H by the construction, and F H denotes the effect of the vertical force applied at H, and having its direction passing through E, the given point; therefore, in this case, the effects of the vertical and oblique forces are the same.

Now, it is manifest from the figure that the chord F *m* is greater than any other chords that can be drawn from the fulcrum F to terminate in the arc *m* D; therefore F *m* is greater than F I; but F I is the representative of the effect produced by the oblique force acting at C in the direction C E, and F *m*, as we have just shown, denotes the effect of the vertical force acting at H in the direction H E; so that the effect of the oblique force may be either greater, equal to, or less than that of a vertical force of the same magnitude, supposing the directions in which they act to pass through the same point.

Having thus demonstrated the conditions that must be satisfied in order to produce a maximum effect, and having shown that the value of the oblique force may be either equal to, greater, or less than that of the vertical force whose direction passes through the same point of the cord, it now remains for us to show in what manner the several equations which we have investigated are to be reduced.

Example.—Let the length of the lever, its weight, and the value of the angle ϕ_n remain, as in the example under equation (H), and suppose a subsidiary force a_0 to be applied at C at the distance of 26·5 feet from A the remote extremity of the lever, and having its direction passing through E at a distance of 44 feet from the point A (see fig. 2 preceding). It is required to determine the value of the force b_0, or the tension upon the lower portion of the cord A E, on the supposition that the required force acting in direction A E, produces the same effect in maintaining the equilibrium, as the force a_0 acting in direction C E.

The first step to be performed in the resolution of this example is to determine the angle F C E $= \underset{0}{\phi}$; and for this purpose we have given the length of the lever F A = 56 feet, and the angle F A P = 32 degrees; therefore, by the rules of plane trigonometry, we have

$$\text{A P} = 56 \text{ sec. } 32^\circ, \text{ and F P} = 56 \text{ tan. } 32^\circ.$$

These lines, however, are not necessary in the calculation; because, from the right-angled triangle A H E, the quantities A H and H E can be found directly from the data, without having recourse to similar triangles: thus we get

$$\text{A H} = 44 \text{ cos. } 32^\circ, \text{ and H E} = 44 \text{ sin. } 32^\circ;$$

the absolute numerical values being $44 \times \cdot 8480481 = 37\cdot 3141164$, and $44 \times \cdot 5299193 = 23\cdot 3164492$ feet respectively. Therefore, by subtraction, it is C H $= 37\cdot 3141164 - 26\cdot 5 = 10\cdot 8141164$ feet; consequently, by trigonometry, we have

$$\frac{\text{H E}}{\text{C H}} = \frac{23\cdot 3164492}{10\cdot 8141164} = 2\cdot 1561122 = \text{tan. } \underset{0}{\phi} = \text{tan. } 65^\circ\ 7'\ 5''.$$

Having thus determined the angle of direction for the subsidiary force $\underset{0}{a}$, the equations by which the actual values of $\underset{0}{a}$ and $\underset{0}{b}$ are to be found are the following, viz.,

$$\underset{0}{a}\, \underset{0}{d} \text{ sin. } \underset{0}{\phi} = \underset{0}{b}\, \underset{n}{d} \text{ sin. } \underset{n}{\phi},$$

$$\text{and } \underset{0}{a}\, \underset{0}{d} \text{ sin. } \underset{0}{\phi} + \underset{0}{b}\, \underset{n}{d} \text{ sin. } \underset{n}{\phi} = \underset{n}{p}\, \underset{n}{d} \text{ sin. } \underset{n}{\phi} = \delta\, w,$$

or, by substituting in this last equation the effect of the force $\underset{0}{a}$ in the first, we get

$$2\, \underset{0}{b}\, \underset{n}{d} \text{ sin. } \underset{n}{\phi} = \delta\, w.$$

Therefore, by restoring the numerical values of the several given quantities, the corresponding value of $\underset{0}{b}$ becomes

$$\underset{0}{b} = \frac{28 \times 2271\cdot 36}{2 \times 56 \times \cdot 5299193} = \frac{567\cdot 84}{\cdot 5299193} = 1071\cdot 56 \text{ lbs.}$$

Here, then, we have a strain of 1071·56 lbs. upon the lower

part of the cord; but by the example under equation (H), we found the strain throughout the cord with only one equilibrating force to be $p = 2143{\cdot}12$ lbs., and this corresponds to the tension on the upper part in the present instance; hence it appears that by the application of the oblique force a_0, the lower part of the cord is relieved of a strain equal to $2143{\cdot}12 - 1071{\cdot}56 = 1071{\cdot}56$ lbs., being just half the original tension; and, consequently, the strength of the cord may be diminished in the same proportion. To find the magnitude of the force a_0 which acts in the direction C E, we must have recourse to the equation

$$a_0 \, d_0 \sin. \phi_0 = b_0 \, d_n \sin. \phi_n.$$

Now d_0, the distance between the fulcrum and the point in the lever where the force a_0 acts, is $d_0 = \text{A F} - \text{A C} = \text{C F} = 56 - 26{\cdot}5 = 29{\cdot}5$ feet; therefore, by substituting the several numerical values in the above equation, the value of a_0 becomes

$$a_0 = \frac{1071{\cdot}56 \times 56 \times {\cdot}5299193}{29{\cdot}5 \times {\cdot}9071767} = 1188{\cdot}23 \text{ lbs. nearly.}$$

Suppose, now, that the magnitude of the force a_0 and the angle of its direction are actually given by the question, instead of being determined by calculation as above, then the value of the force b_0 will be found as follows from equation (K), where we have

$$b_0 = \frac{(28 \times 2271{\cdot}36 - 1188{\cdot}23 \times 29{\cdot}5 \times {\cdot}9071767) \times 1{\cdot}8870799}{56} = 1071{\cdot}56 \text{ lbs.}$$

If the direction of the force a_0 forms with the lever an angle of 90°, then by equation (M) the value of b_0 becomes

$$b_0 = \frac{(28 \times 2271{\cdot}36 - 1188{\cdot}23 \times 18{\cdot}6858836) \times 1{\cdot}8870799}{56} = 1394{\cdot}92 \text{ lbs.}$$

The difference between the two values of b_0 found above is $1394{\cdot}92 - 1071{\cdot}56 = 323{\cdot}36$ lbs., from which the advantage of using the force a_0 in the oblique direction becomes manifest.

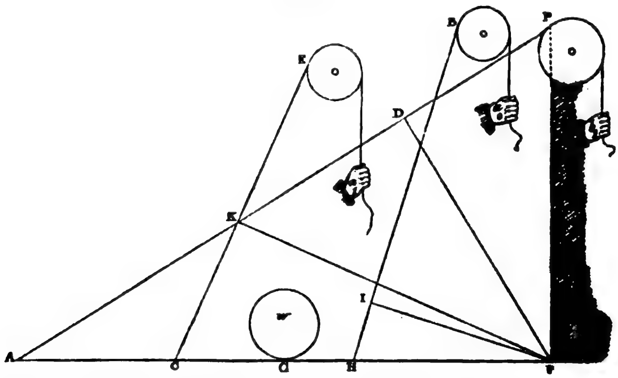
In the preceding process we have taken the value of $d_0 = 18{\cdot}6858836$ feet, being equal to the difference between the whole length of the lever F A and A H; that is, F H$=56 - 37{\cdot}3141164 = 18{\cdot}6858836$; but the same thing may be determined directly from the data by equation (Q), on the supposition that the angle of direction when the force a_0 acts obliquely is known; that is, the value of the force b_0 corresponding to the vertical action of the force a_0 can be determined from the oblique action of a_0 without any previous calculation, by the following formula:

$$b_0 = \frac{\text{cosec. } \phi_n}{d_n} \left\{ \delta w - \frac{a_0 (d_0 \tan. \phi_0 \cot. \phi_n - d_n)}{\tan. \phi_0 \cot. \phi_n - 1} \right\}.$$

An equation which is very easily reduced by substituting the respective numbers as indicated by the combinations.

Let us now suppose that another subsidiary force as a_1 (fig. 6)

Fig. 6.



is added to the system, applied at the point C, and acting

in the direction C E. Then, since the weight of the lever remains the same, as well as the distance from the fulcrum of the point in which it is conceived to be concentrated, it will constantly require the same amount of power to effect the equilibrium, whatever may be the number of forces employed, in whatsoever manner they may be applied, and in whatever directions they may act.

Therefore, in the present case, with two subsidiary forces a_0 and a_1 acting in the directions H B and C E, and the third force b_1 acting in the direction A P, it will require the conjoint effects of all the three to maintain the equilibrium; that is, it will require the effect of the force a_0 in the direction H B, together with the effect of a_1 in the direction C E, and that of b_1 in the direction A P.

Now, by the notation already established, we have the angle F H B $= \phi_0$ and the distance F H $= d_0$, while the angle F A P $= \phi_n$ and the distance F A $= d_n$. In like manner let the angle F C E be denoted by ϕ_1, and the distance F C by d_1; then, if from the fulcrum F the perpendiculars F I, F K, and F D be respectively drawn, and each of them multiplied by the magnitude of its corresponding force, the equation of *equilibrium* arising from their conjoint effects becomes

$$a_0 d_0 \sin. \phi_0 + a_1 d_1 \sin. \phi_1 + b_1 d_n \sin. \phi_n = \delta w. \qquad \text{(R)}$$

Therefore, by reducing this equation in respect of b_1 we obtain

$$b_1 = \frac{(\delta w - a_0 d_0 \sin. \phi_0 - a_1 d_1 \sin. \phi_1) \operatorname{cosec.} \phi_n}{d_n}. \qquad \text{(S)}$$

If the equations (K) and (S) be compared with each other, it will be seen that the former exceeds the latter by the quantity

$$\frac{a_1 d_1 \sin. \phi_1 \operatorname{cosec}. \phi_n}{d_n},$$

for by subtraction we have

$$b_0 - b_1 = \frac{(\delta w - a_0 d_0 \sin. \phi_0) \operatorname{cosec}. \phi_n}{d_n} - \frac{(\delta w - a_0 d_0 \sin. \phi_0 - a_1 d_1 \sin. \phi_1) \operatorname{cosec}. \phi_n}{d_n} = \frac{a_1 d_1 \sin. \phi_1 \operatorname{cosec}. \phi_n}{d_n}.$$

Consequently, the tension on the cord or chain A P, when there are two subsidiary forces a_0 and a_1, is less than when there is only one subsidiary force, as a_0. The cord in the latter case may therefore be made smaller than it is in the former, for since there is less strain there is no necessity for so much strength.

If, instead of supposing the cords, to which the subsidiary forces are attached, to be brought over pulleys and stretched according to the intensity of their respective forces, let them be simply attached to the primary cord or chain A P, as represented in figs. 7 and 8 following; then, if they be stretched to the same degree by means of the load which they are employed to support, the nature of the action will be precisely the same as indicated in figs. 2 and 6, and the whole strain will be thrown upon the point of suspension at P, in the same

Fig. 7.

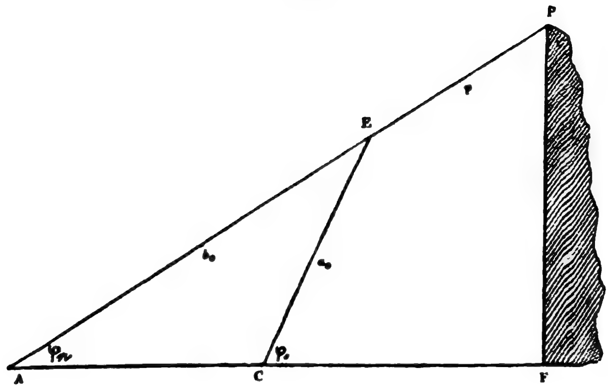
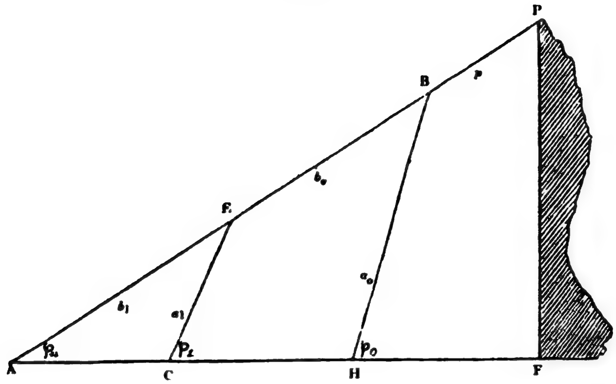


Fig. 8.



manner as if it were transmitted through the suspending bars of a bridge. From these two cases of subsidiary forces, as compared with each other, and also with the case of a single force, the law by which the tension on the cord or chain decreases becomes manifest; it is therefore unnecessary to pursue the investigation further, for by merely extending the notation the successive terms of the series can be obtained from each other by simple induction.

We shall give a tablet of the terms as far as seven subsidiary forces, which we presume will be amply sufficient to enable the attentive reader to detect the law by which the strain diminishes; and this law being once discovered, its application to the construction of suspension bridges becomes a matter of obvious utility. The series of equations denoting the state of equilibrium are as follow:

Force in the direction of the cord or chain	$p\,d_n \sin.\,\phi_n = \delta\,w$
Ditto with one subsidiary force	$a_0 d_0 \sin.\,\phi_0 + b_0 d_n \sin.\,\phi_n = \delta\,w$
Ditto with two subsidiary forces	$c_0 d_0 \sin.\,\phi_0 + a_1 d_1 \sin.\,\phi_1 + b_1 d_n \sin.\,\phi_n = \delta\,w$
Ditto with three subsidiary forces	$a_0 d_0 \sin.\,\phi_0 + a_1 d_1 \sin.\,\phi_1 + a_2 d_2 \sin.\,\phi_2 + b_2 d_n \sin.\,\phi_n = \delta\,w$
Ditto with four subsidiary forces	$a_0 d_0 \sin.\,\phi_0 + a_1 d_1 \sin.\,\phi_1 + a_2 d_2 \sin.\,\phi_2 + a_3 d_3 \sin.\,\phi_3 + b_3 d_n \sin.\,\phi_n = \delta\,w$
Ditto with five subsidiary forces . . .	$a_0 d_0 \sin.\,\phi_0 + a_1 d_1 \sin.\,\phi_1 + a_2 d_2 \sin.\,\phi_2 + a_3 d_3 \sin.\,\phi_3 + a_4 d_4 \sin.\,\phi_4 + b_4 d_n \sin.\,\phi_n = \delta\,w$
Ditto with six subsidiary forces .	$a_0 d_0 \sin.\,\phi_0 + a_1 d_1 \sin.\,\phi_1 + a_2 d_2 \sin.\,\phi_2 + a_3 d_3 \sin.\,\phi_3 + a_4 d_4 \sin.\,\phi_4 + a_5 d_5 \sin.\,\phi_5 + b_5 d_n \sin.\,\phi_n = \delta\,w$
Ditto with seven subsidiary forces	$a_0 d_0 \sin.\,\phi_0 + a_1 d_1 \sin.\,\phi_1 + a_2 d_2 \sin.\,\phi_2 + a_3 d_3 \sin.\,\phi_3 + a_4 d_4 \sin.\,\phi_4 + a_5 d_5 \sin.\,\phi_5 + a_6 d_6 \sin.\,\phi_6 + b_6 d_n \sin.\,\phi_n = \delta\,w$

If we transpose this class of equations in such a manner that the terms involving the quantity b with its several characteristics may stand alone on one side, the effects of the forces on the successive portions of the cord A P, estimated in order from the point of suspension, will become obvious. The transposed equations are as below, having 7 instead of n, that being its numerical value for a system of eight forces.

$$p\,d_7 \sin.\,\phi_7 = \delta\,w$$

$$b_0\,d_7 \sin.\,\phi_7 = \delta\,w - a_0\,d_0 \sin.\,\phi_0$$

$$b_1\,d_7 \sin.\,\phi_7 = \delta\,w - \{a_0\,d_0 \sin.\,\phi_0 + a_1\,d_1 \sin.\,\phi_1\}$$

$$b_2\,d_7 \sin.\,\phi_7 = \delta\,w - \{a_0\,d_0 \sin.\,\phi_0 + a_1\,d_1 \sin.\,\phi_1 + a_2\,d_2 \sin.\,\phi_2\}$$

$$b_3\,d_7 \sin.\,\phi_7 = \delta\,w - \{a_0\,d_0 \sin.\,\phi_0 + a_1\,d_1 \sin.\,\phi_1 + a_2\,d_2 \sin.\,\phi_2 + a_3\,d_3 \sin.\,\phi_3\}$$

$$b_4\,d_7 \sin.\,\phi_7 = \delta\,w - \{a_0\,d_0 \sin.\,\phi_0 + a_1\,d_1 \sin.\,\phi_1 + a_2\,d_2 \sin.\,\phi_2 + a_3\,d_3 \sin.\,\phi_3 + a_4\,d_4 \sin.\,\phi_4\}$$

$$b_5\,d_7 \sin.\,\phi_7 = \delta\,w - \{a_0\,d_0 \sin.\,\phi_0 + a_1\,d_1 \sin.\,\phi_1 + a_2\,d_2 \sin.\,\phi_2 + a_3\,d_3 \sin.\,\phi_3 + a_4\,d_4 \sin.\,\phi_4 + a_5\,d_5 \sin.\,\phi_5\}$$

$$b_6\,d_7 \sin.\,\phi_7 = \delta\,w - \{a_0\,d_0 \sin.\,\phi_0 + a_1\,d_1 \sin.\,\phi_1 + a_2\,d_2 \sin.\,\phi_2 + a_3\,d_3 \sin.\,\phi_3 + a_4\,d_4 \sin.\,\phi_4 + a_5\,d_5 \sin.\,\phi_5 + a_6\,d_6 \sin.\,\phi_6\}$$

The foregoing may be considered as a mathematical illustration of the principle upon which the tapering of the chain depends; but the subject is still involved in difficulty as regards the distribution of the forces, and the position of the oblique suspending rods; for here there is no conditional equation to direct us, and on this account the successful application of the principle to practice, must in a great measure depend upon the sagacity and skill of the engineer by whom the fabric is raised.

In our humble opinion, a system of calculation may be instituted upon the principle of making the angles of direction vary by a constant second difference, at equidistant points along the platform, the limiting angles

being adapted to the circumstances of construction, which must of course be fixed upon before any calculation whatever can be made.

Another condition in the system of calculation might be, to have the effects of all the suspending forces equal; we do not mean the absolute magnitude of the forces, but the effects as referred to their particular directions, when compared with the state of equilibrium; this would manifestly give a series of equal differences for the tensions on the chain A P.

If the several equations above given be successively subtracted from one another, the series of corresponding remainders will stand as below, viz.:—

$$\left.\begin{array}{l}
(p_0 - b_7)\, d\, \text{sin.}\, \phi_7 = a_0\, d_0\, \text{sin.}\, \phi_0 \\
(b_0 - b_1)\, d_7\, \text{sin.}\, \phi_7 = a_1\, d_1\, \text{sin.}\, \phi_1 \\
(b_1 - b_2)\, d_7\, \text{sin.}\, \phi_7 = a_2\, d_2\, \text{sin.}\, \phi_2 \\
(b_2 - b_3)\, d_7\, \text{sin.}\, \phi_7 = a_3\, d_3\, \text{sin.}\, \phi_3 \\
(b_3 - b_4)\, d_7\, \text{sin.}\, \phi_7 = a_4\, d_4\, \text{sin.}\, \phi_4 \\
(b_4 - b_5)\, d_7\, \text{sin.}\, \phi_7 = a_5\, d_5\, \text{sin.}\, \phi_5 \\
(b_5 - b_6)\, d_7\, \text{sin.}\, \phi_7 = a_6\, d_6\, \text{sin.}\, \phi_6
\end{array}\right\} \quad \text{(T)}$$

And if these equations be severally divided by the constant quantity d_7 sin. ϕ_7, the actual values of the differences will be as follows:—

$$p - b_0 = \frac{a_0\, d_0\, \text{sin.}\, \phi_0\, \text{cosec.}\, \phi_7}{d_7}$$

$$b_0 - b_1 = \frac{a_1\, d_1\, \text{sin.}\, \phi_1\, \text{cosec.}\, \phi_7}{d_7}$$

$$b_1 - b_2 = \frac{a_2\, d_2\, \text{sin.}\, \phi_2\, \text{cosec.}\, \phi_7}{d_7}$$

$$b_2 - b_3 = \frac{a_3 d_3 \sin. \phi_3 \operatorname{cosec.} \phi_7}{d_7} \qquad \text{(U)}$$

$$b_3 - b_4 = \frac{a_4 d_4 \sin. \phi_4 \operatorname{cosec.} \phi_7}{d_7}$$

$$b_4 - b_5 = \frac{a_5 d_5 \sin. \phi_5 \operatorname{cosec.} \phi_7}{d_7}$$

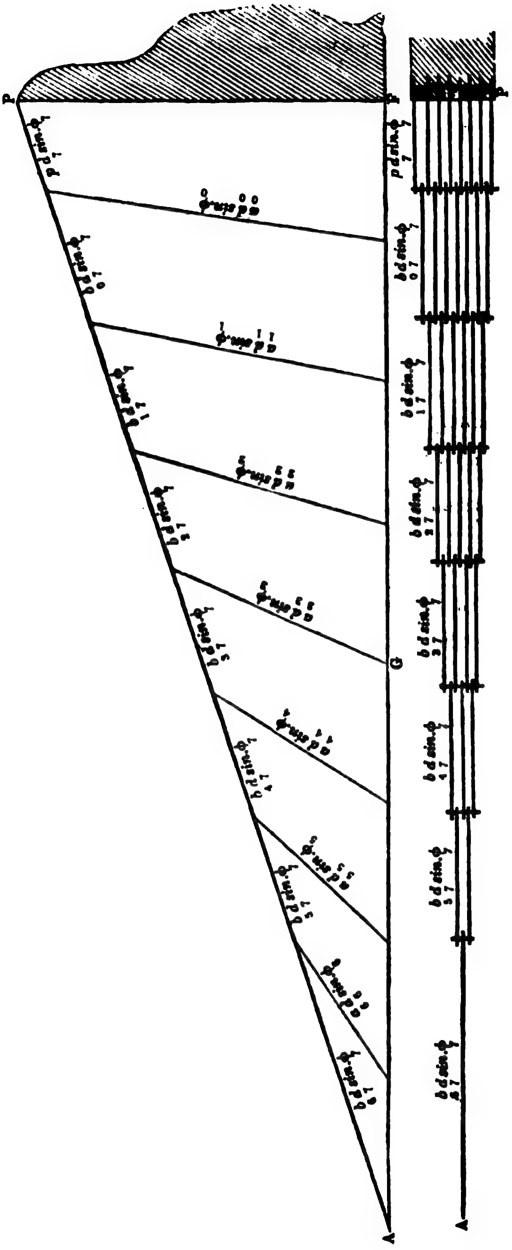
$$b_5 - b_6 = \frac{a_6 d_6 \sin. \phi_6 \operatorname{cosec.} \phi_7}{d_7}$$

Consequently, the difference of the strains upon the first and last portions of the cord becomes

$$p - b_6 = \left\{ \frac{a_0 d_0 \sin. \phi_0 + a_1 d_1 \sin. \phi_1 + a_2 d_2 \sin. \phi_2 + a_3 d_3 \sin. \phi_3 + a_4 d_4 \sin. \phi_4 + a_5 d_5 \sin. \phi_5 + a_6 d_6 \sin. \phi_6}{d_7} \right\} \operatorname{cosec.} \phi_7$$

The following drawing will represent the several quantities involved in the equations, and the tapering plan of the chain will indicate the manner in which the strain diminishes by the application of the subsidiary forces.

ELEVATION.



PLAN OF THE CHAIN.

The following numerical example may be useful in showing how a calculation is to be instituted, supposing that the conditions previously mentioned are admitted as being sufficient to guide the operations of practice in a case of actual construction.

EXAMPLE.—Let A F in the preceding figure be equal in length to 120 feet, and suppose it to be a straight inflexible lever entirely divested of weight and moveable about the fulcrum at F. Let the distance A F be divided into eight equal parts of 15 feet each, and at the several points of division let forces be applied, whose directions make with the horizon angles of 81, 78, 73, 66, 57, 46, 33, and 18 degrees respectively, estimated in order from the fulcrum. Now, suppose a load of 250 tons to be applied at G, the middle of the lever, and to be kept in equilibrio by the simultaneous action of all the forces; it is required to determine the strains upon the different portions of the hypothenuse A P, admitting the several oblique forces acting in their respective directions to produce the same or an equal effect.

Since the load which balances the accumulated effect of all the forces is 250 tons applied at a distance of 60 feet from the fulcrum, the conditions of equilibrium as referred to the property of the lever require that

$$15\,a_0 \sin. 81^\circ + 30\,a_1 \sin. 78^\circ + 45\,a_2 \sin. 73^\circ + 60\,a_3 \sin. 66^\circ + 75\,a_4 \sin. 57^\circ + 90\,a_5 \sin. 46^\circ$$
$$+ 105\,a_6 \sin. 33^\circ + 120\,b_7 \sin. 18^\circ = 250 \times 60 = 15000.$$

ft.

Now, the eighth part of this, or 1875 tons, is the effect of each of the forces in producing the equilibrium; consequently the actual magnitude of each of the forces can easily be found as follows:—

Beginning with the oblique force which is nearest to the fulcrum, and proceeding in order to the most remote, the actual magnitudes of the forces to produce equal effects, accord-

ing to the obliquity of their directions, will be respectively as below:—

$$a_0 = \frac{p\,d_7 \sin.\phi_7}{8\,d_0 \sin.\phi_0} = \frac{1875}{15 \sin. 81^\circ} = 126{\cdot}5581 \text{ tons},$$

$$a_1 = \frac{p\,d_7 \sin.\phi_7}{8\,d_1 \sin.\phi_1} = \frac{1875}{30 \sin. 78^\circ} = 63{\cdot}8963 \text{ tons},$$

$$a_2 = \frac{p\,d_7 \sin.\phi_7}{8\,d_2 \sin.\phi_2} = \frac{1875}{45 \sin. 73^\circ} = 43{\cdot}5705 \text{ tons},$$

$$a_3 = \frac{p\,d_7 \sin.\phi_7}{8\,d_3 \sin.\phi_3} = \frac{1875}{60 \sin. 66^\circ} = 34{\cdot}2074 \text{ tons}, \qquad \text{(V)}$$

$$a_4 = \frac{p\,d_7 \sin.\phi_7}{8\,d_4 \sin.\phi_4} = \frac{1875}{75 \sin. 57^\circ} = 29{\cdot}8091 \text{ tons},$$

$$a_5 = \frac{p\,d_7 \sin.\phi_7}{8\,d_5 \sin.\phi_5} = \frac{1875}{90 \sin. 46^\circ} = 28{\cdot}9617 \text{ tons},$$

$$a_6 = \frac{p\,d_7 \sin.\phi_7}{8\,d_6 \sin.\phi_6} = \frac{1875}{105 \sin. 33^\circ} = 32{\cdot}7871 \text{ tons}.$$

These are the respective magnitudes of the subsidiary forces branching off from the primitive direction A P; and the corresponding magnitudes of the forces acting on the first and last portions of the hypothenuse or chain are as follows, viz.:—

$$p = \frac{\delta\,w}{d_7 \sin.\phi_7} = \frac{15000}{120 \sin. 18^\circ} = 404{\cdot}5086 \text{ tons},$$

$$b_6 = \frac{\delta\,w}{8\,d_7 \sin.\phi_7} = \frac{1875}{120 \sin. 18^\circ} = 50{\cdot}5636 \text{ tons}.$$

Having thus determined the magnitudes of all the subsidiary forces a_0, a_1, a_2, a_3, a_4, a_5 and a_6 together with the magnitudes of the forces which act on the first and last portions of the hypo-

thenuse or chain, it is easy to determine the magnitudes of the forces which act on all the intermediate portions; for, since by the conditions of construction, the effects of all the oblique forces are the same, it is obvious that all the equations in class (U) must be equal to the same constant quantity; that is, the forces acting on the different portions of the hypothenuse or chain, diminish in a regular progression from the highest to the lowest point, and the constant difference may be found from any one of the equations of the class just specified; or it may be found by taking the difference between the forces acting on the first and last portions, and dividing that difference by the number of terms diminished by unity. Now the force on the first portion is 404·5086, and that on the last is 50·5636 tons, and the number of terms or forces is 8; consequently we have $(404{\cdot}5086-50{\cdot}5636)\div 7=50{\cdot}5636$, which is the constant difference sought.

Therefore, if from the magnitude of the force which individually balances the weight, the constant difference be successively subtracted, the remainders will indicate the diminishing forces acting on the hypothenuse or chain; thus we have

$$b_0 = 404{\cdot}5086 - 50{\cdot}5636 = 353{\cdot}9450 \text{ tons.}$$

$$b_1 = 353{\cdot}9450 - 50{\cdot}5636 = 303{\cdot}3814 \text{ ,,}$$

$$b_2 = 303{\cdot}3814 - 50{\cdot}5636 = 252{\cdot}8178 \text{ ,,}$$

$$b_3 = 252{\cdot}8178 - 50{\cdot}5636 = 202{\cdot}2542 \text{ ,,}$$

$$b_4 = 202{\cdot}2542 - 50{\cdot}5636 = 151{\cdot}6906 \text{ ,,}$$

$$b_5 = 151{\cdot}6906 - 50{\cdot}5636 = 101{\cdot}1270 \text{ ,,}$$

$$b_6 = 101{\cdot}1270 - 50{\cdot}5636 = 50{\cdot}5634 \text{ ,,}$$

These forces, however, may also be deduced directly from the formula (K), instead of finding them as above from each other by continued subtraction; and since it may be necessary

on certain occasions to determine them in this way, it will be useful to show in what manner the reduction is to be effected.

By examining the several equations it will readily appear that $d_7 \sin. \phi_7$ in each of them is constant, and is equivalent to $120 \sin. 18° = 37{\cdot}08206$; and the several portions of the entire effect which belong to the respective forces b_0, b_1, b_2, b_3, b_4, b_5 and b_6 are 13125, 11250, 9375, 7500, 5625, 3750, and 1875; so that we have

$$b_0 = \frac{13125}{37{\cdot}08206} = 353{\cdot}9450 \text{ tons.}$$

$$b_1 = \frac{11250}{37{\cdot}08206} = 303{\cdot}3814 \quad ,,$$

$$b_2 = \frac{9375}{37{\cdot}08206} = 252{\cdot}8178 \quad ,,$$

$$b_3 = \frac{7500}{37{\cdot}08206} = 202{\cdot}2542 \quad ,,$$

$$b_4 = \frac{5625}{37{\cdot}08206} = 151{\cdot}6906 \quad ,,$$

$$b_5 = \frac{3750}{37{\cdot}08206} = 101{\cdot}1270 \quad ,,$$

$$b_6 = \frac{1875}{37{\cdot}08206} = 50{\cdot}5635 \quad ,,$$

If we suppose all the values of ϕ but the last, or ϕ_7, to be equal to 90° or a right angle whose sine is unity; that is, if we suppose the subsidiary forces to act in directions perpendicular to the platform or lever, then it is obvious that the actual values of those subsidiary forces will be less than when they act in oblique directions; and for this reason, the several forces which act in the direction of the hypothenuse must be proportionally augmented in order to produce the same effect at the point of suspension; hence the advantage of causing the subsidiary forces to act obliquely; and the same principle applies to the oblique suspending rods in Mr. Dredge's method of constructing bridges.

CONTRACT

FOR ERECTING

A SUSPENSION BRIDGE UPON MR. DREDGE'S PRINCIPLE AT BALLOCH FERRY, LOCH LOMOND.

(See Plate 87.)

It is contracted and agreed between the parties underwritten, viz., Sir James Colquhoun, of Luss, Bart., in the county of Dunbarton, North Britain, on the first part; and James Dredge, of Bath, in the county of Somerset, England, engineer, &c., as principal; and William Gibbons, of Bath aforesaid, maltster, &c., as guarantee, cautioner, and surety on the second part, in manner following: that is to say,—Whereas the said Sir James Colquhoun has resolved to erect a suspension bridge at Balloch Ferry, in the parish of Bonhill, Dunbartonshire, for the transit of foot passengers, horses, carriages, and cattle, conform to plans, elevation, and measurement, designed, drawn, and subscribed by the said James Dredge upon the principle invented by him, which plans consist of Nos. 1, 2, 3, and 4 different parts, which are all subscribed by the said parties as relative to these presents. Therefore, and in consideration of the price hereinafter mentioned, the said James Dredge has bound and obliged himself, and by these presents binds and obliges himself, and his heirs, executors, and representatives whomsoever, to construct and finish the said bridge, of sufficient strength, and in a good and workmanlike manner, conform to the plans, elevation, and measurement above mentioned; subject always to such alterations as shall appear to the said James Dredge to

be rendered necessary for the proper strength and durability of said erection.

And the said James Dredge obliges himself, at his own proper costs and charges, to provide the whole requisite working drawings and sections, models for masonry and iron-work, and also to furnish all iron rails and timber, and all other materials whatsoever (masonry materials excepted), which shall be necessary and fit to be used in or about the said bridge, and to superintend the erection of the same, and of the whole mason-work therewith connected.

And more particularly the said James Dredge binds and obliges himself, and his foresaids, to erect and finish the said bridge in manner following, viz.: of twenty feet width, and two hundred feet in length from the centre of one tower to the centre of the other, and of not less than seventeen feet and one half foot in height above the level of the River Leven at high flood, or at least as high from the surface of the river as the Bonhill Bridge, with an opening of forty feet outside of each tower. The platform of the bridge to be three inches thick, of larch timber, and the other parts of good iron and good workmanship, with iron gates, and the whole of the iron-work to be once well painted; and in general, without prejudice to the particulars above specified, the said James Dredge binds and obliges himself, and his foresaids, to construct and finish the said bridge and openings conform to the foresaid plans, elevation, and measurement, with such alterations as may be rendered necessary as aforesaid; and that the whole of the said bridge shall be executed in the best and strongest manner, and of the best materials.

And the said second parties hereby warrant that the said bridge, when completed, shall be capable of sustaining in transit double the weight of any load, whether of cattle, carriages, or otherwise, that it may ever be fairly exposed to. And further, the said James Dredge binds and obliges himself, and his foresaids, to proceed to the execution of the work above mentioned, and to carry on the same without any intermission, and completely finish the whole within two calendar months immediately after the completion of the towers of the said bridge. Providing hereby and declaring, that if the said

James Dredge shall fail so to complete the said bridge within said period of two months, he shall be bound to allow to the said Sir James Colquhoun, and the latter shall be entitled to retain from the price after specified, an abatement of five per cent. upon the amount which the said Sir James Colquhoun shall have advanced for masonry on the said bridge, and that during the time the said bridge shall remain unfinished after the expiring of the foresaid two months.

And it is hereby agreed that if the fencing off of a foot-path upon said bridge shall be dispensed with by the said Sir James Colquhoun, he shall in that case be allowed a deduction of twenty pounds from the stipulated price of said bridge hereinafter mentioned; and in the event of his requiring only two gates on said bridge in place of three shown on said plan, he shall receive a further deduction on that account of five pounds sterling. And further, if any other part of the said plans shall be dispensed with by the said Sir James Colquhoun, or not executed by the said James Dredge, the expense of such part or parts shall also be deducted from the price after mentioned. And if any of the conditions or particulars above specified shall be altered or executed contrary to the foresaid plans, these shall not be construed into an abandonment of the said plans, but the same shall be completed, and the present contract shall be effectual notwithstanding such alterations.

And further, the said second parties, viz., the said James Dredge, as principal, and the said William Gibbons, as cautioner, guarantee, and surety for him as aforesaid, bind and oblige themselves, conjunctly and severally, and their foresaids, to warrant, uphold, and maintain the said bridge and towers thereof in good and sufficient condition for the space of five years after completion thereof, excepting from this obligation any injury or damage which shall arise thereto by lightning, earthquake, civil commotion, or malicious damage, or not fairly within the compass of wear and tear by ordinary use of such bridge. And if during the said period of five years any parts of said towers or bridge be discovered to be insufficient or requiring repair (not rendered necessary from the causes aforesaid), then the second parties shall be bound and

obliged, as they do hereby bind and oblige themselves and their foresaids, to repair such parts and make them sufficient at their own expense immediately on the insufficiency being discovered. For which causes, and on the other part, the said Sir James Colquhoun binds and obliges himself, his heirs and successors, immediately on such bridge being completed and proved to be sufficiently strong in all its parts as after mentioned, which proof shall be ascertained within one calendar month after its completion, to make payment to the said James Dredge, or his heirs, executors, or assigns, of the sum of one thousand five hundred pounds sterling, as the agreed price of the said bridge to be constructed by the said James Dredge in terms hereof, including therein the whole of his charges for engineering, travelling, and all other charges and expenses competent to him for said work, excepting such reasonable charge for superintending the erection of the said towers, either by himself or by a qualified person appointed by him, as may be considered fair between the parties, or as shall be fixed by reference to a respectable tradesman resident in Dunbartonshire, in case they do not agree. It being here contracted and agreed that the said Sir James Colquhoun shall, when required by the said James Dredge, furnish the timber requisite for said bridge, and be entitled to retain out of the said price of one thousand five hundred pounds the current price of such timber so furnished, besides the expense of dragging, conveyance, and whole workmanship thereof; and on the completion of said bridge, the price of said timber and the said expenses attending the same (which last shall be ascertained and proved by the account and statement of the wood forester on the estate of Luss) shall be deducted from the said stipulated sum of one thousand five hundred pounds; and the balance thereof, after such deduction, shall then be immediately payable to the said James Dredge and his foresaids. Providing always that the said bridge, including mason-work, shall have been previously duly tested and proved, within the aforesaid period of one calendar month, to be of the requisite strength, and completed in all respects according to agreement, and in such a manner as to afford to the public a safe com-

munication across the river Leven. But it is hereby expressly provided and declared that, if between the completion of the said bridge and the expiration of said five years, the bridge, including towers, shall not have fairly borne its work, and be thus proved unable or insufficient to answer the purposes for which the bridge is intended, the said James Dredge, as principal, and the said William Gibbons, as guarantee, cautioner, and surety foresaid, shall be bound and obliged, as they hereby bind and oblige themselves and their aforesaids, conjunctly and severally, to repair and make the bridge substantial in every respect, or to make repayment to the said Sir James Colquhoun, his heirs and assigns, not only of the foresaid sum of one thousand five hundred pounds sterling, but also of the further sum of two hundred pounds as and for a moiety of the outlay and expense disbursed by the said Sir James Colquhoun in the erection of the mason-work connected with the said bridge; and on such repayment, the said James Dredge and his foresaids shall be fully entitled to the whole of the iron and wood-work of the said bridge as his own property, and all the materials excepting the stone and mason-work thereof. And it is further hereby provided that, in the event of any difference arising with respect to the true meaning of the present contract or the execution of any part of the work hereby contracted for, the parties hereby submit the same to the final determination of William Steele, Esq., Advocate, Sheriff Substitute of Dunbartonshire, and failing him, to the Sheriff Substitute of the said shire for the time being, as sole arbiter, and oblige themselves and their foresaids to abide by and fulfil any decision which he shall pronounce on the matters hereby submitted to him. And both parties bind and oblige themselves and their foresaids to implement and perform their respective parts of the premises to each other under the penalty of five hundred pounds sterling, to be paid by the party failing to the party observing the contract or willing to do so, besides performance. And both parties consent to the registration hereof in the Books of Council and Session Sheriff Court of Dunbartonshire or other Judges' Books competent for preservation; and that letters of horning, or six days' charge, and all other

execution needful, may hereon pass on a decree to be interponed hereto, and for that purpose constitute

Procurators; in witness whereof these presents, written upon stamped paper by James Mackibbin, Clerk to Robert Grieve, Writer, Dunbarton, are subscribed,

(L. S.)

(L. S.)

(L. S.)

SPECIFICATION OF THE QUANTITIES OF MATERIAL

USED IN THE

SUSPENSION BRIDGE AT BALLOCH FERRY,

DUNBARTONSHIRE.

PLATE 87 is an isometrical projection of a suspension bridge now in course of erection across the Leven at Balloch Ferry in Dunbartonshire, for Sir James Colquhoun, of Luss, Bart., after designs and under the superintendence of Mr. Dredge, of Bath.

The whole extent of the suspended road-way is 292 feet; but as a space is left on each side as an easy access to and from the towing-paths, the distance from the centre of one pier to the centre of the other is reduced to 200 feet, which must be considered as the span of the bridge.

Each pier for supporting the chains consists of two octagonal towers upon which the chains rest, and which are connected together by springing a light arch across the road-way. These towers taper from 15 feet 2 inches by 9 feet 2 inches at the base, to 9 feet by 3 feet at the top, the longest dimension being in the direction of the bridge. The entire height of the towers is as under, viz.

	feet.
From bed of river to water-mark	3
,, water-mark to upper surface of road-way .	16
,, road-way to tops of towers	21
	40 feet.

The distance from the centre of one octagonal tower to the centre of the other across the road-way is 20 feet, equal to the width of the bridge. The towers and the key-stone of the arch are built of a reddish freestone from Bonhill; the stringing courses at the road-way, springing of the arch, and the offsets at the top of the tower, of a white stone in the neighbourhood.

Upon the tops of the towers are large cast-iron plates, on which the chains rest.

The principal chains for supporting the structure are formed of $\frac{7}{8}$ round bars, laid side by side to the number of 13 upon the tops of the towers, and successively dropping one at each joint until it arrives at the centre, where that part of the chain is reduced to a single bar. The links upon the towers are 6 feet long, but after springing from thence they are increased to 9 feet.

1. The weight of each link, 6 feet long, is 14·5 lbs.

The weight of iron on the towers is therefore
$14{\cdot}5\,(13 \times 4)$ . . . . $= 754$

The weight of each link, 9 feet long, is 20 lbs.

The weight of iron is therefore $20\{8(12 + 2 \times 5{\cdot}5) + 4\} = 12400$

$(24 \times 4) = 96$ connecting bolts increasing in arithmetical progression from 3 to 12 lbs. each,
$\frac{96}{2}(3 + 12)$ . . . . $= 720$

Weight of iron in chains . . $= 13874$ lbs.

The length of the oblique suspending rods, if radius remained constant, would be as the sec. of the angle which they respectively make with the horizon; but in consequence of the inclination of the chains towards the joints of connexion with the suspending rods, their length diminishes continually towards the centre of the bridge; and as the ascertaining the exact length of each rod separately would involve much complicated calculation, we will avoid it, and assume (which will be very near the truth) the average length of the bars to be 11 feet 9 inches, (excepting those immediately proceeding from

the joints on the towers:) the two bars from each joint of the chains being ¾ of an inch in diameter, we have, for the weight of the whole of the ¾ bars,

2. $*1{\cdot}47\ (88 \times 2 \times 11{\cdot}75)$	=	3039·96
16 oblique rods from the towers ⅞ diameter, averaging 26·6 feet in length:		
$*2\ (16 \times 26{\cdot}6)$	=	851·2
136 pieces ⅞ rod, each 2 feet long, welded to the end of the oblique rods:		
$2\ (136 \times 2)$	=	544·
136 nuts, each ·25 lb.		
$\cdot25 \times 136$	=	34·
56 pieces 1 inch in diameter, 2 feet long, in retaining rods:		
$2{\cdot}61\ (56 \times 2)$	=	292·32
56 nuts, each ·333 lb.		
$\cdot333 \times 56$	=	18·64
152 nuts and pins, each ·5 lb.		
$\cdot5 \times 152$	=	76·
Total weight of oblique suspending rods	=	4856·12 lbs.

Proceeding onwards, we next arrive at the horizontal beams, or those running from end to end of the bridge; and there are here 2 of 190 feet long, and 4 of 45 feet long each, all 5 inches × ½ inch, so that

3. $8{\cdot}4\ \{(190 \times 2) + (45 \times 4)\}$	=	4704
64 connecting plates, each 1 foot long:		
$8{\cdot}4 \times 64$	=	537·6
6 ⅝ pins to each plate, each ·5 lb.		
$\cdot5 \times 64 \times 6$	=	192
Total weight of horizontal beams		5433·6 lbs.

We have now to examine the weight of the transverse beams, or those which run at right angles to and immediately support

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* Where 1·47 and 2 are respectively the weight in lbs. of a ¾ and a ⅞ rod 1 foot long.

the platform; they are each 20 feet long, 5 inches deep × ½ an inch in thickness, and have at every 6 inches along their length, near the upper edge, holes of $\frac{11}{16}$ths of an inch drilled to admit of ⅝ rods passing longitudinally through them from end to end of the bridge; there are 117 such beams in all:

4. 8·4 (20 × 117) . . . . . = 19656

Every third beam is trussed with suspension trusses, consisting of 5 rods, viz., 1 = ⅞ and 4 = ⅝, each 6·66 feet long:

2 (39 × 6·66) . . . = 519·48
1·02 (39 × 4 × 6·66) . . =1059·739

1579·219

$\frac{1579 \cdot 219 \times 8}{100}$ . . . . = 126·337

1705·556 = 1705·55

21361·55

From which deduct for the quantity of iron not used in the holes,

$1 \cdot 23 \left(\frac{40 \times 117}{24}\right)$ . . . . . = 239·85

21121·70

The ⅝ rods to which we before alluded run 6 inches apart through the transverse beams from end to end, and are placed there for the purpose of combining the whole mass together, and causing a more equable pressure upon the beams which support the road-way; there are, therefore, 40 rods of 195 feet long each, and 80 of 45 feet, so that the whole weight will be

5. $1 \cdot 02 \{(195 \times 40) + (80 \times 45)\} = 11628.$

There are also 28 bars, each 10 feet long, 1⅛ in diameter, to be used as holdfasts in the ground for retaining the chains.

6. 3·31 (28 × 10) . . . . . . = 926·8

28 nuts, ·75 lb.

·75 × 28 . . . . = 21

947·8 lbs.

And by adding the results together we shall have

1 . .	= 13874
2 . .	= 4856·12
3 . .	= 5433·6
4 . .	= 21121·7
5 . .	= 11628
6 . .	= 947·8

		tons.	cwt.	qr.	lbs.
Total	57861·22=	25	16	2	13·22

the whole of the wrought-iron actively employed in the structure; to which, if we add that used in railing,

2 (400 × 2 × 4)	= 6400
and about 400 lbs. more as staples, &c. . .	400
	6800 lbs.

tons.	cwt.	qr.	lbs.
3	0	2	24
25	16	2	13·22
28	17	1	9·22

the total quantity of wrought-iron used in the bridge.

We have next to examine the cast-iron used; and beginning with the plates on which the chains rest,

1. Four plates, each 560 lbs. . =2240

136 circular plates for fastening the oblique rods to the horizontal beam, each 3·5 lbs.

2. 136 × 3·5 . . . . . = 476

22 lbs. cast blocks to each truss, and there being 39 trusses:

22 × 39 . . . . . = 858

2 cast boxes for each beam for attaching it to the horizontal longitudinal beams, each 11 lbs.

11 × 2 × 117 . . . . =2574

28 castings in retaining chains, each 5 lbs.

5 × 28 . . . . . = 140

	tons	cwt.	qr.	lbs.
6288 lbs.=	2	16	0	16

as the total quantity of cast-iron actively employed in the bridge; to which add the quantity used in the railing,

	tons	cwt.	qr.	lbs.
$28 \times 4 \times 35 = 3920 =$	1	15	0	0
	2	16	0	16
	4	11	0	16

The timber used in its construction is larch, grown on the estate of Sir James Colquhoun at Luss, and the quantity used is,

$292 \times 20 \times \cdot 25$ . . . . . . $= 1460$

2 beams of wood passing from end to end, 6×3 inches:

$2\ (292 \times \cdot 5 \times \cdot 25)$ . . . . . . $= \quad 73$

1533 feet.

The whole of the material used is,

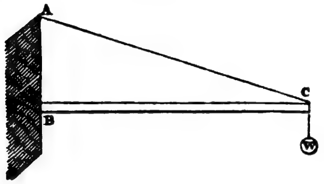
	tons	cwt.	qr.	lbs.
1st, Wrought-iron	28	17	1	9·22
2nd, Cast-iron	4	11	0	16
3rd, Timber	1533 cubic feet.			

The principle upon which this bridge is built is entirely new; and it has been doubted by many whether the arrangement of the bars in the chain and the adoption of the oblique suspending rods is a judicious application of iron in the construction of such bridges. The small quantity of material used in the one specified above, (which is intended as a permanent structure for general traffic,) is however, a sufficient proof; and we shall endeavour to show by mathematical reasoning, that it is more than equal to sustain any load which it is possible to place upon the surface of its road-way.

In calculating the force of a bridge of this description, the formulæ involved are necessarily very different from those used in estimating the common catenary; for here one half of the bridge balances the other, the structure being held in equilibrio entirely by the acting chains, and the centre part of the chain where it is reduced to a single link, may be severed or entirely removed without affecting the stability of the structure. It is therefore evident that such a bridge may be

referred to the projection of two brackets, and estimated accordingly.

Fig. 1.



Conceive the weight W to be supported by a cord A C at the extremity of a horizontal projection B C, and maintained there by a tension in the line A C, and compression in the line B C; then A B C being a right angled triangle by construction,

$$\sin.\ ACB : \text{rad.} :: AB : AC,$$

$$\text{or } AC = \frac{\text{rad.}}{\sin.\ ACB} \cdot AB = AB.\ \text{cosec.}\ ACB$$

by the principles of projection.

$$AB : (AC) = AB.\ \text{cosec.}\ ACB :: W : W\frac{\text{cosec.}\ ACB\ .\ AB}{AB} =$$
$$W\ \text{cosec.}\ ACB.$$

Let w=W, the weight resting on the point C.

$\theta = \angle ACB$, contained by the horizontal line and the line A C.

x=the tension in the line A C (caused by the weight W) in the direction C A.

y=pressure or thrust in the direction C B.

Then, by substituting these quantities for the former proportion, we have

$$\sin.\ \theta : \text{rad.} :: w : x,$$

and by equating the products and reducing the equation, we get

$$x = \frac{\text{rad.}}{\sin.\ \theta} w = w\ \text{cosec.}\ \theta \quad . \quad . \quad . \quad . \quad . \quad (1)$$

And for the pressure or thrust in the direction C B, it is

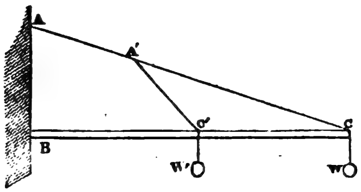
$$\sin.\ \theta : \cos.\ \theta :: w : y;$$

consequently by reduction, we have

$$y=\frac{\cos.\theta}{\sin.\theta}w=w\cot.\theta \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Which equation holds true for every value of the angle θ; but there is the same strain in every part of the line A C, and if such were to remain, there would be no occasion to increase the size of the chain towards the point A. But let us further suppose another weight W′ to be supported by a line from some intermediate point between B and C, and let that line be attached to some point A′ in the cord A C, as in fig. 2.

Fig. 2.



Allowing A C B to remain as before, and supposing the same quantities to retain the same values as in the former figure, and putting in this w', θ', x', y' respectively for the weight W′, the < A′ C′ B, the tension in the line A′ C′, and the pressure in the direction C′ B, we have, for the value of x' by equation (1),

$$x'=w'\ \text{cosec.}\ \theta' \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The point A′ in the line A C is acted upon by two forces represented by the symbols x, x', the resultant of which forces in the direction towards A will be,

$$R=\sqrt{x^2+x'^2+2\,x\,x'\cos.(180°+\theta-\theta')} \quad . \quad . \quad . \quad (4)$$

The former values of x and x' being substituted, we get

$$R=\sqrt{w^2\text{cosec.}^2\theta+w'^2\text{cosec.}^2\theta'+2w\,\text{cosec.}\theta\,.\,w'\cos.\theta'\cos.(180°+\theta-\theta')}$$

And the difference will be represented by

$$D=\sqrt{w^2\text{cosec.}^2\theta+w'^2\text{cosec.}^2\theta'+2w\,\text{cosec.}\theta\,.\,w'\cos.\theta'\cos.(180°+\theta-\theta')}$$
$$-w\ \text{cosec.}\ \theta,$$

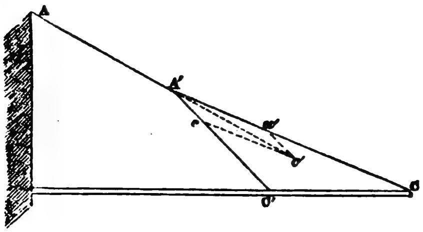
and in which proportion, the chains should increase in sectional area towards the base. Proceeding in like manner for another section, the resultant of the forces would be

$$R'' = \sqrt{(x''^2 + R^2 + 2\,R\,x'' \cos.\,(180^\circ + \delta - \theta'')}* \quad . \quad (5)$$

From which it appears, that each progressive section of the chain towards the point of suspension, has an increase of strain, equal to the resultant of the two forces acting at the further extremity of that section towards the extremity of the bracket; and from this it is evident, that the chain should taper in the same proportion; but it is equally clear, that the use of the oblique suspending rods is peremptorily insisted upon, otherwise the reduction of material towards the extremity of the bracket beyond a certain extent would be a proportionate reduction of power.

The following diagrams will graphically illustrate this:

Fig. 3.



Draw $a'\,c'$ (fig. 3) parallel to A′ C′, produce A A′ onwards to the point c', and the triangle of forces A′ a' c' being complete, it only remains that the lines supporting the brackets should be increased in power proportionally to the length of the sides of the triangle to which they are respectively parallel.

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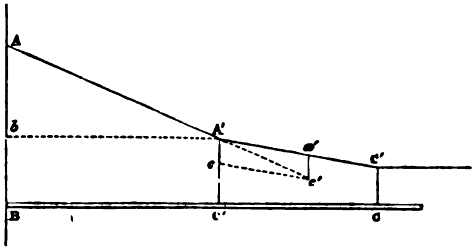
* Where x'' represents the value of the tension sustained in the angle or inclination θ'', R the resultant of the first two forces, and δ an angle of which R is equivalent to the cosec.

Fig. 4.*



Again (fig. 4), draw a' c' parallel to A′ C′, and continue A A′ to c'; then will the triangle of forces be complete, as in the former curve; but it is evident by the construction of the bracket, that the pressure of the horizontal force is not sustained by the road-way. Draw A′ *b* parallel to B C, then will A′ *b* be the measure of that horizontal force, or equal to the weight resting on A′ drawn into the cotangent of the angle A A′ *b*, which must be counteracted by a sufficient power in the direction A′ C′; and the weight of the iron, which would be equivalent to that power, would be oppressing the chain in the direction A′ A, as the cosecant of the angle A A′ *b*.

It is well known that the parameter of the catenary of a suspension bridge, is equivalent to half the weight of the structure drawn into the cotangent of the angle which a tangent to the curve at the point of suspension forms with the horizon, and the oblique strain is equal to half the weight drawn into the cosecant of the same angle.

Therefore, from what has been said, it is clear that a parallel section of iron, equivalent to the parameter of the catenary, may be taken away from one end of the curve to the other, and that not only the weight would be saved, but also another

* I would here presume that whilst speaking of a bracket supported by a catenary curve with vertical suspending rods, a tensile force must be acting from the point C′ equivalent to the parameter of that curve.

quantity of iron which was necessary to support it. These, then, are the advantages which arise from the use of the oblique suspending rod.

Example.

Suppose A B C (fig. 1) to be a bracket projecting from a wall with a weight of 364 lbs. attached at the point C, the angle A C B being 39°; required the tensile power upon the cord A C, and also the horizontal pressure in the direction B C?

By equation (1) we have,

$$x = w \text{ cosec. } \theta,$$

therefore,

$$364 \times 1{\cdot}589015 \quad . \quad . \quad . \quad . \quad . \quad . \quad = 578{\cdot}401 \text{ lbs.},$$

and by logarithms,

$$\begin{array}{lrl} \text{log. } 364 \quad . \quad . & = & 2{\cdot}561101 \\ \text{log. cosec. } 39° & = & 10{\cdot}201128 \\ \hline \text{nat. num.} & & 2{\cdot}762229 \quad = 578{\cdot}401 \text{ lbs.} \end{array}$$

As shown by equation (2),

$$y = w \text{ cot. } \theta,$$

hence

$$364 \times 1{\cdot}234897 \quad . \quad . \quad . \quad . \quad . \quad . \quad = 449{\cdot}502 \text{ lbs.},$$

by logarithms,

$$\begin{array}{lrl} \text{log. } 364 \quad . \quad . & = & 2{\cdot}561101 \\ \text{log. cot. } 39° \quad . & = & 10{\cdot}091631 \\ \hline \text{nat. num.} & & 2{\cdot}652732 \quad = 449{\cdot}502 \text{ lbs.} \end{array}$$

Again, let every thing remain as before; but suppose another weight equal to 364 lbs. were added to the point C′, as in fig. 2, and supported at an angle of 50°; required the increase of strength from A′ towards A, and also the horizontal pressure towards B?

By equation (3),

$$x' = w' \text{ cosec. } \theta';$$

consequently,

$$364 \times 1{\cdot}305408 \quad . \quad . \quad . \quad . \quad . \quad = 475{\cdot}16 \text{ lbs.}$$

$$\begin{array}{lrl} \text{log. } 364 \ . \ . \ . & = & 2{\cdot}561101 \\ \text{log. cosec. } 50^\circ \ . & = & 10{\cdot}115746 \\ \hline \text{nat. num.} & & 2{\cdot}676847 \quad = 475{\cdot}16 \text{ lbs.} \end{array}$$

Therefore 475·16 lbs. is the tension in the direction C′ A′; but to ascertain the value of the tension from A′ to A, by referring to equation (4) we find,

$$R = \sqrt{x^2 + x'^2 + 2\,x\,x' \cos.\,(180^\circ + \theta - \theta')},$$

and by substituting the numerical values for the several quantities, we get

$$R = \sqrt{578{\cdot}401^2 + 475{\cdot}16^2 + 2 \times 578{\cdot}401 \times 475{\cdot}16 \times {\cdot}981627}$$

that is

$$\begin{array}{rl} 578{\cdot}401^2 & = 334547{\cdot}7168 \\ 475{\cdot}16^2 & = 225777{\cdot}0256 \\ 2 \times 578{\cdot}401 \times 475{\cdot}16 \times 981627 & = 539567{\cdot}0241 \\ \hline \end{array}$$

and

$$\sqrt{1099891{\cdot}7665} = 1048{\cdot}75 \text{ lbs.}$$

The required strength in that part of the cord A A′, and the difference between the part A A′ and A′ C is,

$$\begin{array}{r} 1048{\cdot}75 \\ 578{\cdot}401 \\ \hline 470{\cdot}349 \end{array}$$

And the sectional area of the one part should be to the sectional area of the other as

$$1 : 1{\cdot}8131.$$

For the magnitude of the horizontal force; that which is occasioned by the weight at the further extremity of the bracket has been shown to be = 449·502 lbs.; but the adding of another weight at C′ will cause an increase towards B, so that the aggregate of force from C′ to B is represented by

$$449{\cdot}502 + (364 \times {\cdot}839100) = 754{\cdot}934 \text{ lbs.}$$

And the difference of pressure between the part C′ B and C C′ is

$$754{\cdot}934 - 449{\cdot}502 = 305{\cdot}432.$$

Example 2.

Suppose a figure similar in every respect to fig. 4, and let a weight of 364 lbs. be attached to the horizontal line from the point C, and let A′ C′ form the same angle with the horizon that A′ C′ did in the former curve, viz., 39.° Then the tension in the direction A′ C′ will be

$$364 \times 1{\cdot}589015 = 578{\cdot}400 \text{ lbs.}$$

and the horizontal force, as before represented, is equal to

$$364 \times 1{\cdot}234897 = 449{\cdot}502 \text{ lbs.},$$

which must in this case be resisted through the cord in the direction from C; or, in short, as the parameter of the catenary; and therefore, the absolute tension in the direction C′ A is in consequence, equal to 1·589015 (364 + the weight of material requisite to sustain a pull of 449·502 lbs.), and which, in large spans, increases to such an enormous extent as to facilitate the destruction of the structure.

The throwing the thrust on the towers or abutments into the horizontal line is of advantage to the road-way, as rendering it more steady and not so liable to that oscillation to which suspension bridges are commonly exposed; and in practice, the sectional area of the beam to sustain the transverse action of the platform, would be more than sufficient under any circumstance, to overcome the horizontal action to which the structure may be exposed: for that reason, therefore, it would be useless to increase the beam towards B.

The preceding examples which refer to the bracket only are strictly arbitrary, and have been proposed for the purpose of showing in what manner the formulæ arising from the investigation are to be applied; but as a further illustration of the advantages to be derived from the adoption of the system just

discussed, we shall revert to the bridge across the River Leven for which the quantity of materials has already been ascertained. The whole quantity of material employed is apparently very small, and if compared with that which would be required on the old principle, might tend to create a doubt as regards the strength and stability of the structure. In order, therefore, to remove any suspicions that may be entertained on that point, we shall here endeavour to show that the material used, though small in quantity, is amply sufficient to sustain a much greater load, than under any possible circumstances can ever be brought upon the platform.

There are four points of suspension, and at each point there are twelve bars in the chains besides two oblique suspending rods, all of $\frac{7}{8}$ths of an inch in diameter; this gives a section of 8·4185 square inches at each tower or point of support, or, in all, 33·674 square inches of section. Now, since the absolute tensile power of a $\frac{7}{8}$th bar of malleable iron of a medium quality is 35000 lbs. very nearly, the absolute strength of the iron at the joints adjacent to the towers is 875 tons, for

$$(12=2) \times 4 \times 35000 = 1960000 \text{ lbs.} = 875 \text{ tons.}$$

In the next place, we have to ascertain the greatest tension, that under any circumstances can come upon the chains at the points of support, and for this purpose we shall suppose the weight of the platform, chains, diagonal rods, &c., between the towers to be 67200 lbs., and taking the greatest extraneous load to which a bridge ought ever to be exposed at 120 lbs. per square foot, we get

$$190 \times 20 \times 120 = 456000 \text{ lbs.}$$

the length of the platform being 190 feet; therefore we have

$$67200 + 456000 = 523200 \text{ lbs.}$$

for the total weight to be supported.

Now, the angle which the first series of bars would make with the platform, if continued downwards till they intersect it, is 24 degrees, and the natural cosecant of 24 degrees, is 2·458591; consequently, by equation 1 (page lv), we obtain for the effect of obliquity,

$$523200 \times 2{\cdot}458591 = 1286334{\cdot}8112 \text{ lbs., or } 574{\cdot}2566 \text{ tons.}$$

Let this be subtracted from the absolute power of the iron at the points of suspension, and we have

$$875 - 574{\cdot}2566 = 300{\cdot}7434 \text{ tons,}$$

which is the surplus strength in the chains after the platform is fully loaded; and every part of the structure, if properly tested, would be found equally sufficient for the intended purpose.

The preceding method of obtaining the surplus strength of the bridge is simple and easily applied, but it may be done otherwise as follows.

We have found that the entire weight to be supported in an extreme case is 523200 lbs., and we have moreover seen, that the ultimate strength of a single bar of the iron used, is 35000 lbs.; consequently, the number of bars required at the towers will be

$$523200 \div 35000 = 14{\cdot}948 \text{ bars.}$$

This, however, is on the supposition that the iron is strained vertically in the direction of its fibres, but this in the case of a bridge is contrary to fact, the bars being all placed in an oblique position with respect to the horizon, while the weight acts in a direction perpendicular to it; the power of the iron must therefore be reduced in the ratio of radius to the sine of the angle of obliquity; or, which is the same thing, the weight must be increased in the ratio of the cosecant to radius. Now the angle of obliquity is 24 degrees, and its cosecant 2·458591; hence we get

$$523200 \times 2{\cdot}458591 \div 35000 = 36{\cdot}75 \text{ bars;}$$

but there are 56 bars at the towers in the Balloch Bridge; consequently we have

$$56-36{\cdot}75=19{\cdot}25 \text{ bars},$$

or 4·8125 bars at each tower for the surplus strength, even in the case of an extreme load.

ESSAY AND TREATISES

ON THE

PRACTICE AND ARCHITECTURE

OF

BRIDGES.

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AUTHOR OF TREATISES IN THE ENCYCLOPÆDIA BRITANNICA ON ARCHITECTURE AND BUILDING, &c. &c.

PREFACE.

The Author of the following Essay and Treatises begs it to be remarked that they have been written sequently as they appear; and as the composition has been spread over more than three years, and the continued research and study which the subject demanded have been followed by new ideas on some matters, and perhaps better information on others, discrepancies may occur between the earlier and the later portions of the work. These, however, can be but slight, and need not detract in any degree from the usefulness of the work if it be taken as a whole,—greater weight being always given to a later than to an earlier expression of opinion when any difference may appear, or be found, to exist.

The Author claims for himself originality in the suggestion of groining a bridge arch, or of carrying a groining through the length of a series of arches, and which he believes may become of great importance in the composition of bridges;—he believes himself entitled also to whatever credit may be due for suggesting and showing in what manner a bridge may be further improved and economized by placing the parapets upon a corbelled cornice, and generally for the evidence he has produced to show that piers may be greatly reduced in thickness and the bays extended, without detracting from the sufficient, or from even the apparent, strength of such a work as a bridge.

The Author desires it to be understood, however, that he claims no credit, and acknowledges no responsibility, for any thing in the present publication, or in the engraved drawings

of various bridges attached to it, but what is contained in the immediately preceding Preliminary Essay, and Practical and Architectural Treatises, with their wood-cut illustrations, and the illustrations contained in the Plates numbered 88,[1] 19, and 39, these being respectively Perronet's design for bridges at Melun,—longitudinal sections of the river under the central arch of Old London Bridge, from Smeaton,—and the central arch of London Bridge, as it is, and arranged to show the longitudinal central groining and corbelled parapet cornice, suggested in the text, applied in such a work.

The Author would beg to state further that the delays in the production and publication of the parts of which this volume is made up are wholly attributable to him, and not in any way to the Publisher, whose patience has often been wearied past endurance by the enforced procrastinations of the Author, who has been able to devote to the work only such time as he could spare, and that often occurring at long intervals, from the more active duties of professional life.

WILLIAM HOSKING.

London, October, 1842.

[1] It will be observed that the numbers of the Plates have in some few instances been altered.

PRELIMINARY ESSAY.

A Bridge is a constructed platform supported at intervals, or at remote points, for the purpose of a road-way over a strait, an inlet or arm of the sea, a river or other stream of water, a canal, a valley or other depression, and over another road, and is distinguished from a causeway, or embanked or other continuously supported road-way, and from a raft, by being so borne at intervals, or at remote points. Constructions of the nature and general form and arrangement of bridges, such as aqueducts and viaducts,—the former being to lead or carry streams of water or canals, and the latter to carry roads or railways upon the same or nearly the same level over depressions,—are in practice considered as bridges, although they are not such in the commonly received sense of the term. Taken, however, in the sense which the most plausible etymology that has been suggested of the term[1] would require,—the word *bridge* being formed by

[1] "Skinner's conjecture, that the final part of the word, sc. *rige*, is the Anglo-Saxon *rige, hricg*, a ridge,—appears worthy of notice. It

prefixing the constructive *be* to *ridge,*—a bridge is an elevated construction upon or over a depression and between depressed points. The bridge of the middle ages, or of the period when the term bridge was formed to indicate the object, was indeed what the term appears to indicate, a constructed ridge. Arched in one span, or with the effect of an arch of one span,—the outer arch including a series of arches, of sizes adapted to fill it, or to produce its form,—the old bridge of the European nations, from the decadence of the Roman empire nearly until the present time, springs from the low margins of a stream, and rises to a high summit over it; fully justifying the term applied to designate it as an object; and that term change and custom have made applicable to objects of various form, proportion, mode of arrangement and construction, but serving the same purpose and answering the same end.

These remarks are with reference to the English term *bridge;* but what we so designate was made and used, in some form or other, ages before the peculiar construction alluded to had given rise to that or to any other now existing term.

Etymological and antiquarian speculations and researches are not to the purpose of the present work,

accounts for the application of the word to the bridge of the nose, and to the bridge of a violin. In Anglo-Saxon we find *hricg, bricg;* in Swedish, *rygg, brygga;* in German, *ruck, bruck;* in Dutch, *rugge, brugge;* in English, *ridge, bridge.* The common prefix *be* supplies the only difference between the two words in each language."—*Encyclopædia Metropolitana, Art.* Bridge.

further than may be necessary to make the terms applied clearly intelligible, and to develope the practice of bridge-building in all cases which may tend to elicit the best mode or modes of practice to attain any particular end. Whether the *Γέφυρα* of the Greeks, or the *Pons* of the Latins, had any more definite application to the bridge, or mode of constructing bridges, of the Greeks and Romans respectively, than the English word BRIDGE has to the varieties of composition and construction that the present practice of bridge-building involves, it would be irrelevant to inquire, and impossible to determine. If *pons* be derived from *pendeo*, to hang in the air, it may be held to intimate that the Romans made suspension bridges, and probably with tendrils or thongs, before they acquired the art of building such structures as the bridges which remain to the present day, of their work. If, however, *pons* be from *pono*, to place or lay down, it may be that the stepping-stones of a brook gave origin to the term, since applied, in the sense of the English term bridge, to the magnificent works which still attest the power and give evidence to the skill of the extraordinary people who constructed them. The Greek term may, in like manner, be tortured into presumed components, to show why and how it applies as a term to what we designate a bridge, although lexicographers treat *Γέφυρα* as a primitive word; but no useful end would be answered by pursuing such speculations further in this place.

A bridge, taken as a word and not as a mere term, is simply and substantively a thing by which a difficulty

may be surmounted and got over, and the works which we term *bridges* are physical constructions, artfully contrived for getting over physical obstructions, or difficulties of a certain kind; the difficulties themselves being, in a great number of cases, only as it regards peculiar conveniences, or arising from the use of artifices which require peculiar facilities to make them available. A streamlet, that may hardly be called a brook, being in a gully over which a man may step, stops the way to a carriage upon wheels, and thereby impedes the progress of the explorer, or of the migrating colonist with his bullock-waggon, and intercepts the march of an army with its artillery. The rude temporary bridge which may give the means of overcoming the difficulties interposed by a brook to such parties as those supposed, and who require it but for the occasion, is found insufficient when the settler has to take the productions of his farm, or of his flocks and herds, to market by the same route, or when the country on both sides has become subject to the same military command. Permanence has then become essential to the value of the bridge, as a means of overcoming the difficulty occasioned by the brook; but commerce requires that facility of approach and passage shall be added to permanence, and joins with luxury and refinement in demanding also that to these shall be superadded beauty of material and elegance of design. To pursue this case further;—commerce and its concomitants may be supposed to have refused to remain satisfied with the bridge by which the brook is passed

over, though it combine comparative ease of approach with convenience of passage and agreeableness of design, and to demand that the whole depression or valley shall be obliterated in its effect upon the road, by a bridge from one summit to the other; so that commerce may carry on its business, and luxury pursue its pleasures, at the speed of a bird in the air.

In like manner it is required, that in addition to facility of passage over and upon a bridge, there shall be facility of passage under it; and not for the stream merely, but also for the commerce of men and merchandise which are borne along upon the waters. Bridges are made to obviate this difficulty, and the social and commercial intercourse of communities and nations pass along at the same time and with equal facility and convenience, in their different modes, upon the waters of a river, strait, or estuary, as the case may be, and upon the road which a bridge carries over it.

Such speculations as the foregoing might be extended almost indefinitely; but it may be enough to have called attention to this consideration, that a bridge is not of one particular form, size, proportion, material, mode of construction, arrangement, or design, but that it is such as circumstances require it to be. It may therefore be desirable to point out generally the circumstances that arise in which a bridge becomes necessary, or may be required, before entering upon the modes of answering the demand, and meeting the various exigencies that will arise.

The first and most obvious circumstance that dictates the necessity of a bridge is the occurrence of a stream or

body of water in the line of a great public road; such obstruction to the continuity of a road being otherwise either physically, politically, or economically impassable: though, indeed, it is of frequent occurrence that when the necessity exists, other contingent circumstances make it similarly impracticable to construct a bridge. A torrent may be so spread upon a wide flat bed as to be easily fordable; but if it be confined within a deep ravine or gully it may be called physically impracticable for a road without a bridge; as a river, estuary, strait, or arm of the sea, being wide and having low banks, may be both politically and economically impracticable for an erected bridge, where the navigation for vessels with lofty rigging is to be kept free: whilst it may be economically practicable by means of a passage raft or punt, or as such a contrivance is commonly called, in the more general sense of the term *bridge*, a floating bridge. Floating bridges or ferries will indeed be often found sufficient for the demand of a particular line of road where no political obstruction exists to the erection of a bridge to carry the road over; the width of the passage being supposed so great as to make the latter economically difficult, if not impracticable. It must not be overlooked, however, that the use of a road is greatly affected by the facilities which it presents to the traveller; and the fact that a floating bridge or ferry is not fully employed, affords no proof that the public service does not require that the road should be made continuous by the erection of a bridge, even when the ferry appears to be more than enough for the demand.

The necessity of keeping the navigation of a river or other water-way open for loftily rigged sea-going vessels may be sufficient to prohibit the erection of a bridge where it would be otherwise highly desirable. The great mail-road from the metropolis through Bristol into South Wales is intercepted by the Severn; the river being wide, its banks low, the water sufficient for marine navigation, and the trade of the country requiring that its course should remain open for that purpose, the erection of a bridge is almost absolutely prohibited, as none could be built but in such manner and at such cost as to make it economically impracticable.

High banks and the comparative narrowness of the channel, in both cases, rendered it practicable in every point of view to pass a road over the Wear at Bishops' Wearmouth, by Sunderland, without intercepting the navigation of the river; and, in like manner, the Straits of the Menai are passed over by the great Holyhead road without impeding the passage of ships through the Straits.

Bridges are, indeed, as various as the circumstances that demand them, and under which they are executed. There are large classes of works, however, answering for and bearing the name of bridges, that must be excluded from a treatise like the present, which is limited to the arts of designing and executing permanent constructed erections as bridges. Bridges on boats or on rafts, as ferry bridges and works of that class, require to be separately treated of, whilst the various expedients used by the military engineer for facilitating the passage of troops with their baggage and

the materials of war across rivers, or other obstructions of the same kind, must continue to form a separate study; for the more common want is that which it is desired here to supply.

Constructed erections as bridges are formed of various materials; and these are, for the most part, timber,—the practice of combining which is known as carpentry; stone,—the working and setting of which, or rather the result of the working and setting stone in construction, is called masonry;[2] brick,—which is known in composition as brick-work; and iron,—which is prepared by the founder, and fitted and fixed by the smith, the result of their joint labours being designated simply, iron-work.

Although a bridge may be built almost entirely of timber, as indeed bridges often are built,—except as to the smith's work in the form of shoes, rings, hoops, straps, bolts, nuts, washers, screws, spike and other nails, which the carpenter finds essential to the proper and efficient combination of his principal material,—the best timber bridges are those in which solidity and evenness of pressure, with power of resistance and retention, are

[2] It is desirable in matters relating to building, as well as in most others, that terms should be exclusively applied to what they best define, describe, or intend. Masonry, taken as it is commonly defined, "*ars cæmentaria*," must include building with bricks, as well as working and building with stone, or their results in execution; but this extensive application of the term is inconvenient, and it will be restricted in this treatise to what is best understood by it,—cemented constructions of the natural substances known as stone,—whilst brick-work will be used to describe cemented constructions of the artificial material, brick.

given by piers and abutments of masonry or brick-work; and, in like manner, what is called an iron bridge may be said to require that its piers and abutments, or other points of support, shall be of masonry, or of a combination of mason's work and brick-work. Bridges are built, and in some cases most efficiently so, of brick-work alone; but, that a brick bridge may be durable and sightly, it is almost always necessary, and it is always desirable, that some of the more exposed parts should be of stone. Masonry may stand alone in the composition of a bridge, but neither brick-work nor masonry alone, nor the two in combination, can be made to effect such objects as may be attained by the aid of the iron-founder and smith, and, indeed, of the carpenter with his timber.

Circumstances will, however, and they frequently do, dictate the material of which a bridge shall be built. The locality will often point out the material by furnishing one particular kind and denying others, and the use of that kind economy will very generally impose, unless, indeed, the end to be answered cannot be attained without the aid of some other.

Of all the materials used for bridge-building, timber is the most extensively and variously available; it is the material with which most can be effected at the smallest cost, except in very extraordinary localities and under such peculiar circumstances that the exceptions need hardly be taken into account here. Besides being itself directly applicable as a material for bridge-building, it is almost absolutely essential as an auxiliary in erecting a bridge of any magnitude with other materials, so that

wherever a bridge may be built of masonry or brick-work, according to the ordinary practice in making such constructions, a bridge might be made of the timber necessary to assist in executing the work with brick or stone.

Timber is, nevertheless, the most liable, of all the materials named, to destruction from natural decay, and it is more exposed than stone, brick, or iron, to injury from accident and from incendiarism. Besides the liability of timber to early decay when exposed to the weather and to alternations of wet and dry, it is also subject to change in its bulk; in the first instance in becoming seasoned, or as the natural moisture of vegetation dries out, and constantly swelling or shrinking afterwards, as it becomes wet or dry, until decay shall have destroyed its absorbent powers. Changes in bulk occasion changes in the form of the work composed of a material subject to such incidents, and a timber bridge is therefore exposed to comparatively early destruction from that cause alone.

These remarks apply to timber generally; but it is certain that some sorts are less obnoxious to the objections stated to its use, as a material component of a bridge, than others are; and moreover, expedients are resorted to for the purpose of protecting timber from some of the causes of decay, and in so far making it less objectionable, whilst modes of combining timber which shall make it less liable to be affected by hygrometric changes, and modes of composing timber bridge constructions to afford the work protection from the weather, have been proposed, and as the ends to be

answered are important, such proposals deserve to be fully considered.

Iron, as the main constituent of a bridge, possesses many valuable qualities;—its tenacity and power of resistance in a comparatively small body allow of the construction of works, and the production of effects with it, which are impracticable and unattainable with any other available material. But the good qualities of iron as the main constituent of a constructed bridge, are much affected by the changes which take place in its bulk upon the access and abstraction of heat, together with its susceptibility to chemical changes, and by the most ordinary agents. It is thus more unstable in framing, and more actively injurious to what it may thrust against and be connected with, than timber, whilst its substance wastes away by oxidation, or it becomes changed by other processes into what is incapable of performing the duties that had been intrusted to the unimpaired iron.

Exposed as the principal parts of a bridge must be to meteoric influences, it will, perhaps, be found impossible to obviate altogether the injurious effects of the changes in bulk, or of the expansion and contraction of iron when used in large ribs, braces, and beams; but they may be counteracted in a great degree by judicious arrangements in framing and combining the parts of an erection of the kind alluded to. Iron may also be protected to some extent from the canker of oxidation by simple and renewable processes; and experience and observation have taught us to avoid bringing iron into

contact with those influences which have appeared to operate injuriously upon it.

As an auxiliary in bridge-building, iron is second in value to timber alone; and indeed timber and iron may be called co-essential auxiliaries, although neither may enter into the composition of a structure. Still there are but few works of magnitude and importance into which both timber and iron do not, almost of necessity, enter; but their introduction and application in such cases as are alluded to, demand the greatest prudence and foresight, that their inherent defects may be prevented from coming into injurious operation, and that their valuable qualities may be properly educed and applied.

Stone is, however, pre-eminently the bridge-builder's material. The carpenter can supply the want of a bridge in a comparatively short time, and, in most cases, at a small cost; and the smith and founder will, with moderate assistance from the mason or bricklayer, effect what cannot be done with stone, and will, in some cases, supply the place of stone with iron where stone might be used; but grandeur of effect, power of resistance, and eternity of endurance, are to be sought in masonry, in the mason's art, and with the mason's material.

The qualities which make stone so peculiarly fit for the purposes of the bridge-builder are, its incompressibility and inflexibility,—which may together be expressed by the one word unyieldingness,—its massiveness, or the circumstance of its being obtainable in blocks or pieces of large size, and its plastic virtue, or its capability

of being cut or wrought to any form, size, or figure, together with its retentiveness of the form given to it. A further excellence in stone as a material for permanent constructions, and especially for bridges, is its almost entire freedom from tendency to change in bulk through meteoric influences, and more particularly from the presence or absence of moisture or of heat. Indeed, the substance of the sorts of stone fittest for bridge-building is almost inaccessible to moisture, or rather the stone is good in the ratio of its inaccessibility to such agent, whilst the expansion and contraction of stone from the access or egress of heat are so small as to have been considered, till very lately, quite inappreciable; but in so far as the susceptibility is appreciable, it is a defect.[3]

We know of and possess nothing, however, as a material for massive permanent constructions, and fitted for bridge-building particularly, so free from liability to change in bulk, from any natural influence, as stone; and nothing, therefore, considering its other qualities as essentials, so well adapted for the main constituent of a bridge.

The practical inelasticity of stone, which prevents it

[3] Mr. Rennie, a son of the eminent Engineer-Architect of Waterloo Bridge, has remarked that the heading joints of the coping course of the parapets over the haunches of every arch of that bridge indicate, by alternately opening and shutting in winter and summer, the access or abstraction of heat in the mass of the structure of the arches. Professor Mahan, of the Military Academy of the United States of America, intimates that the influence of heat upon stone in producing the effects alluded to, had been remarked by Professor Bartlett, of the same establishment.

from being disturbed by concussions, is also to be cited in its favour for the purpose alluded to, but with that it possesses a quality which limits its availability in some important particulars. Being readily frangible, stone can only be used and applied where it shall not be subjected to transverse strain. It is unfit therefore for beams, or to bear across over a void, and especially where the situation or service exposes it to disturbance from concussions, unless it be with a bulk altogether disproportioned to the length borne over; but it should be understood and received as a principle, that the quality or characteristic referred to renders stone unfit to be trusted with a transverse strain under any circumstances.

The inelasticity and frangibility of stone altogether prevent the combination of parts by framing, as may be done with timber and iron, so as to give greater strength or power of resistance to parts than was possessed by the whole. A stone may indeed be made to carry a greater weight across or over a void when cut up into pieces of certain forms, if peculiarly arranged, as in an arch, than it might be trusted with or would carry across the same span in the form of a beam or lintel; but the parts so arranged would require the restraint of an extraneous tie, or a loading of the abutments and haunches, which were not necessary to the level bearing across, and which a susceptibility of being framed, or rather of acting with effect as framing, would have obviated.

The foregoing remarks apply to stone generically; but although the qualities mentioned are those which make stone pre-eminently the material for permanent erections

of the nature of bridges, there are others which are essential to render any sort of stone properly fit for use in the construction of a bridge, and others again which are necessary to render a sort economically or otherwise available. These, as well as the sorts of work, or modes of applying stone in construction, will be discussed in connexion with the subject of masonry as applied in bridge constructions.

Brick, for the purposes of bridge-building, can only be considered as a substitute for stone, and will be used only when and where stone of proper quality is either physically or economically unattainable. Brick, like stone, is incompressible and inflexible, but it is prepared, almost of necessity, in small, regular, and equal forms, and these are incapable of being changed in shape or figure to any useful purpose. If well made and thoroughly burnt, brick seems to be quite free from liability to expand or contract under any influences to which it can be exposed in a bridge; but this material cannot be used as the main constituent of any work without a large proportion of a foreign plastic substance to bed and pack together into coherent constructions the infinite number of minute forms which brick as a building material exhibits. This substance, under the name of mortar or cement, is, from its preparation and composition, yielding and changeable, until it has set and become perfectly dry, which does not generally occur for some time after it has been applied. How hard and enduring soever mortar or cement may become after a time, no mortars or cements known, and in use, are in the first instance, nor do they become for a

long time, if ever they do, equal to the brick in power of resisting pressure and the various actions to which bridge constructions are exposed. Hence, and because of the equal and rectangular form given to the brick in ordinary use, an arch turned with uncut bricks is dependent entirely upon the mortar or cement with which the bricks are packed, that substance being inserted of necessity to make up the difference between the lengths of the inner and outer peripheries of the ring, or part of a ring, formed by a course of bricks in an arch. Thus it will appear, that an arch turned with uncut bricks must be liable to change its form, and consequently to become insecure, while the mortar or cement remains compressible, or less hard and unyielding than the substance of the brick.

These are inherent defects in brick-work as ordinarily practised, and it is not pretended that any advantage would be derived from making bricks larger than they are usually made, or by making them in forms adapted to arches; for to both expedients there would be serious objections, and the defects alluded to could hardly be affected in a sensible degree by either. No brick could be made well and with economy that should be of a much larger than the ordinary size, and no brick could be made, in like manner, for the large arches which are contemplated when a bridge is spoken of, long enough to render it unnecessary to make up the substance requisite to give an arch of any magnitude sufficient strength by a plurality of rings; unless indeed bricks were moulded to the various forms and sizes necessary to bond a large arch, as in the gauged arch of finished

house-building, which would be altogether out of the question.

The porousness of brick, which alone is enough to render brick-work highly defective in all hydraulic constructions, is a further objection to that material in such bridge constructions as expose the work to the access of water; and these are of constant occurrence. Brick is not, however, necessarily porous in so great a degree as the bricks usually produced are found to be; but when brick is made more compact, it is more costly; and if bricks are subjected to any process to close the pores, the surface ceases to receive the adherence of the usual mortars and cements.

But, as before remarked, with skill and care, and with good materials, in many cases and under many conditions excellent constructions as bridges, and of the nature of bridges, are built, and may be built, of brick-work, or, better still, with brick and stone, by a judicious combination of the two; stone being introduced in chains and strings, to assist in bonding the brick-work, and in springing, blocking, and coping courses, and upon salient and exposed parts generally, and to receive and distribute pressure where it is greatest. Rigid adherence to the rationale of construction would, indeed, prohibit the combination in parallel positions of any two sorts of material, as brick and stone, which offer their substances of such widely different sizes and incoherent shapes, that no sure result can be counted upon, because of the greater quantity of compressible packing material, as mortar, that must be introduced with the one than the

other requires. Thorough bonding courses,—whether in vertical constructions or in arches,—springing, blocking, and coping courses, are, however, not only unobjectionable, but they may have the effect of materially modifying the defects which do and must exist in brickwork as the main constituent of a bridge.

This general view of the merits of the principal materials of which bridges are built, and of the causes, properties, and qualities which affect them in practice, is intended to mark the leading characteristics of each in composition, and not to enter into explanatory details that are not necessary to a fair appreciation of them all; but a few observations may be proper, to show why a defect that may be obviated may yet constitute an objection to a particular class of materials, and why what is an objection to one class of materials may not be so to another.

It has been remarked as an objection to timber in permanent constructions, that it changes its bulk by shrinking as it becomes seasoned. Now, it is obvious that this may be met in a great degree by seasoning the timber before it is worked; but bridges are built of timber for economy and dispatch, and both of these would be materially affected in almost every case by insisting upon the use of properly seasoned timber; so that the objection may stand for what it is worth, or be put aside as inapplicable, according to the circumstances of each particular case. Again, it is a broad objection to brick-work, that so large a proportion of what should be an entire unyielding mass in construction, is a yielding compressible substance; and this objection is stated to be

stronger as it regards the construction of arches in brick-work, because the rectangular form of the brick renders it necessary that the value of the arch-form in construction should be derived from the insertion of mortar in the triangular wedge-formed spaces which are left between the bricks in an arch. This may be met by cutting the bricks; but bridges are built of brick for economy, and it would be an endless task to do this for a large work; besides that, it would in reality be of very little value when done, as before intimated, since there must still be a body of compressible mortar in every joint, to fill inevitable interstices and to bed the bricks, whilst the shallow rings of which a large brick arch must be composed would still remain unbonded together. The repetition of the brick in rings, each ring outwards holding more solid matter than that within it, practically meets the difficulty, though it does not remove it, nor does it bond the rings to make every one of value to all the rest.

The objection to brick-work for bridge-building, because of the mortar required in its composition, may be held to apply to masonry also; but in well-proportioned and well-executed upright and horizontally coursed masonry it is an objection little more than in name, as the infrequency of the joints, and the truth with which the sides or beds of the stones are wrought, requiring but a thin stratum of mortar to seat and joint the stones, the character of such construction for unyieldingness ought not to be sensibly affected by the practical necessity for mortar in its composition. Nevertheless, it should be

borne in mind that a tendency to imperfection, through this practical necessity, does exist, and it may be taken as a rule in masonry setting that there should be no more mortar in any joint than enough to prevent the absolute contact of the stone, that the air may be excluded, and an even bearing secured. In large arches the tendency to imperfection which the existence of mortar in the radiating joints occasions, becomes a positive defect, which is felt in a greater or less degree according to the quantity of mortar in the joints. In this, then, consists the broad distinction between brick and stone in the composition of a bridge arch with reference to the employment of the yielding and compressible substance, mortar. In a rough brick arch mortar is essential to the formation of the arch,—that is, to give the form and to obtain the virtue of an arch,—and in a stone arch mortar is only necessary to take the place of the air which the unpolished surfaces would otherwise allow to circulate in the joints; the service performed by the mortar being the mechanical one of excluding the air, and thereby promoting the cohesion of the separate blocks.

It may be further remarked, that quick setting mortars or cements, which are, for the most part, much less yielding than ordinary mortar, may be used with bricks in brick-work, and their use certainly lessens the objection to brick-work in respect of the mortar; but it makes an inroad upon economy, and besides, quick setting mortars are brittle, and are therefore themselves not unobjectionable in such a work as a bridge arch, where the weight

of the materials cannot be brought to bear and to produce its effect in compression until the centering is removed, when fractures are produced where common mortar would have accommodated itself without material detriment to its future usefulness in the work. In works of hewn masonry, quick setting cements cannot be used with propriety at all, because of the time that must elapse in and between the spreading of the mortar, and the bedding or setting of a heavy stone in its place upon it.

In the foregoing remarks no account has been taken of rubble masonry, as it can hardly be admitted at all in the construction of a bridge, that sort of work being liable to all the objections which the use of an indefinite quantity of a compressible and yielding substance irregularly mixed with a hard and rigid material in shapeless masses can occasion. Excellent work for many purposes may be produced with rubble masonry, but it is not to be included in the masonry adapted for bridge-building as a matter of choice or selection. Strong and adhesive mortars will enable the mason to construct what shall have the form and serve the end of an arch, if it be of small size and quick sweep; but unless the work is made by dressing and shaping the stones to bear over all with the power of resistance of the main constituent, the result can only have the strength of the mere mortar, whatever that may be. There are circumstances, indeed, under which a bridge of sufficient strength and durability for what is required may be formed of concrete, but such would not be a construction, nor can an arch of rubble

masonry be considered but as a better sort of concrete formation. It must be distinctly understood, however, that the objection to placing dependence upon mortar rests upon the presumption that it is, as mortars usually are, less capable of resistance to pressure, and that it is in other respects less trustworthy than the stone or brick set in it.

Such being the leading characteristics in composition of the usual main constituents, or of the principal component materials of the different classes of bridge constructions, the next consideration is, under what circumstances any, and what one, may be adopted for application in preference to any other.

It may be stated as a general rule,—liable, of course, as all general rules are, to exceptions,—that wherever the object to be attained in the use of a bridge can be effected with timber, the same end may be answered better and more effectually, both for use and duration, with iron; and wherever and whenever the objects proposed in the use of a bridge can be attained by the use of brick, the same thing can be done better and more effectually, in every sense of the word, and for every purpose, with stone;[4] whilst iron and stone are available to the accomplishment of objects that are un-

[4] It seems almost needless to state, that by the term 'stone' is intended quarried stone of proper quality for the use of the mason in construction; but it may be necessary to explain, that it is here intended to include those sorts of stone only that will bear greater pressure without crushing than ordinary mortars and cements, or, indeed, than the ordinary brick used in building will bear.

attainable with timber and brick respectively. Wherever, again, a choice exists between iron and stone, the latter will deserve the preference, though, as it has been already remarked, there are many objects attainable by the use of iron that cannot be accomplished with stone.

But the uses of life require bridges for the mere ends they answer, and the best that can be obtained must often be accepted where a better would be desired. Timber and brick will answer the commercial demand of society as efficiently, for the time, as iron and stone; and the art of constructing a bridge in the readiest, simplest, and cheapest form, will be found, in the great majority of instances, more valuable than that of erecting the most magnificent and the most durable.

Economical considerations will thus seek to control, as they will often direct, what a required bridge shall be made of. An infant or scattered community would act unwisely in expending its energies in the erection of costly bridges whilst their lands remained uncultured and their flocks and herds untended. Bridges may be supposed necessary to give value to the products of industry, but these must be adapted, in the cases supposed, to supply the want with the smallest expenditure of labour; the works being executed, nevertheless, rather in advance of the existing condition of the community, that the benefit the works themselves will confer may not leave them behind the improved condition of the community. Bearing in mind, then, the merits of the various classes of construction, as characterized by their main constituents, the economical condition of the party,

company, or community, and the end to be answered, the engineer-architect will estimate and advise, not with reference to what he would desire, but to what is, under the circumstances, properly required. Giving their due weight to those considerations which affect the durability of a structure of the kind contemplated, the aim should be rather to counteract injurious tendencies in the cheaper materials, than to make their existence an excuse for adopting the more expensive unnecessarily, while it must not be overlooked that the conservation and repairs, with the tendency to early destruction, of the cheaper, may more than counterbalance the original cost of the more expensive but more enduring class of construction.

Where the navigation of a water-way is of great importance, it may be proper to make it a consideration in advising or determining upon the class of construction to be adopted in building a bridge over it, whether one or another will allow of the erection with least interruption to the navigation; and it may be a part of the economical consideration, how far the repairs essential to the conservation of the structure can be effected from time to time without occasioning interruption to the traffic upon the bridge itself, as well as to the navigation under it. The same may be said, indeed, in the case of a road over which another road, a railway, or a canal, is to be carried; interference with one channel of social commerce must be taken into account in providing the means of another, and to that must be added the consideration of its liability to recur.

It can seldom happen that it will not be desirable to

make a bridge with the smallest possible number of points of support, seeing that by far the greatest proportion of the contingencies to be provided for and against are most conveniently disposed of by an arrangement to that effect. Piers in a water-way intercept the current and impede the navigation; they are most troublesome and expensive to found and form, and are most exposed to injury when they are formed. The object of the bridge itself—a convenient road over—being properly provided for, and the permanence of the structure being sufficiently considered, it is not too much to say that the aim and end of the bridge-builder should be to reduce the piers to the smallest possible number consistently with a due regard to economy. Of all the bridges over the Thames at and near London, the Suspension Bridge at Hammersmith interferes least with the navigation of the river, and is the least exposed to injury from the action of the current upon its points of support, these having, to a certain extent, the effect of embankment walls, which prevent the stream from spreading itself uselessly, if not injuriously, over a wide shallow bed, and direct the current upon the mid-channel, whereby it is kept free and clear; whereas the lumbering masses which support Putney Bridge obstruct the navigation, force the current into narrow rapids which tend in every way to the destruction of the works themselves, and make the passage upon the river dangerous. The laden barge of commerce and the double-banked barge of pleasure pass with or against the stream, and alike with ease and safety, under the tasteful and scientific erection,

which carries a convenient and agreeable road over, and leaves the water-way uninterrupted, while both are exposed to inconvenience and danger where the ugly piles of Putney and Battersea support narrow and inconvenient road-ways over the dammed-up river. In like manner, the effect of Southwark Bridge, with its two well-formed piers of neatly executed masonry, is hardly felt upon the river, whilst the multitude of awry-looking, angular piers of Vauxhall Bridge, standing across a bend of the river, are with difficulty avoided by the heavy craft, which depend almost entirely upon the current for motion.

These are timber and iron bridges of various forms and modes of arrangement; and with the materials of which they are composed no sensible inconvenience, and much less obstruction, should in any case have been imposed upon the navigation of the river. That neither obstruction nor inconvenience is necessary with even a bridge of masonry is shown by the New London Bridge, which contrasts advantageously, not alone with its predecessor, but with all the other stone bridges upon the river, in these respects. The infrequency of the piers, and their moderate bulk, together with the expanse and elevation of the arches, preserving the head-way almost unabated over a great part of the whole width of the water-way, show in these, as in other respects, an example of the highest degree of perfection in the practice of bridge-building. It is the results, however, that are here referred to in contrast, and not the means by which they were produced. The expanse of water-way, and

the excellence of the head-way underneath, and the easy inclination of the road-way upon the bridge, as well as the general magnificence of the work, are due to circumstances that did not exist in the cases of the other bridges, none of which possess all those qualities and in a like degree. Had Labelye and Mylne built with granite, their works could not have been executed with the funds at their disposal respectively, or at any rate for the sums expended upon them in their erection; but with granite, and the means of applying it, they would possibly, or they ought to, have occupied less of the water-way with obstructions; and with the means at their disposal of raising the approaches, they would probably have avoided making the road-ways upon their bridges so steep as to be always inconvenient and sometimes dangerous.

The level line of road-way upon Waterloo Bridge, and the noble effect of the level line and equal arches in the elevation, were purchased at a very high price in the cost of the approaches which permitted them to be attained; and while Blackfriars Bridge furnished a standard or gauge for head-way to vessels navigating the river, unlikely to be at any time removed or altered,[5] it might not be thought worth the abandonment of those advantages and beauties to obtain greater height under any of the arches than the greatest given by Blackfriars Bridge. Nevertheless, that fewer piers and a greater breadth

[5] The removal of Old London Bridge was in contemplation, and was treated as a certainty, when Waterloo Bridge was designed, and Southwark Bridge did not then exist.

between some of them, with a correspondingly expanded head-way, would have been better for the navigation, is clearly shown by the greater facility with which it is carried on within the 700 feet of water-way under London Bridge, than under Waterloo Bridge with one half more; and not only is the navigation easier, but the run of the water is less severely felt under the former than under the latter of these two bridges. Next to the effect of position as to the river itself, these results must be attributed to the less frequent recurrence in London Bridge of the obstruction that a pier forms both to navigation and to the stream.

It will be found, too, that the abutments of London Bridge run out into the bed of the river for a considerable distance from each bank, and thus force the body of water into less than would appear to be its natural channel. This might be expected to act in the manner of a partial dam; but it does not so to any injurious extent, because the water being retarded along the sloping banks of a river by friction against and upon the banks, its tendency is, in a considerable degree, to run from the banks to the middle and *vice versâ*, as the tide may be ebbing or flowing, rather than in the direction of the current of the main body. A stream, and particularly a tidal stream, may therefore be contracted by its margins without materially increasing the force of its current, whilst an insulated pier placed within the range of the stream, clear of the margins, is, for so much of the space as it occupies, a dam to the stream, as well as an obstruction to the navigation.

Upon this view it may be argued that it would have been better if the abutments of Waterloo Bridge had been advanced or projected in upon the bed of the river, to the diminution in number of the arches, and the increase in span of the reduced number. The channel being thus narrowed, the water would act upon the bottom with more effect than it does at present, and probably clear the shoals which contract the efficient water-way of the river in the bend where that bridge occurs. It is true that wing walls, or even embankment walls, might in such case become necessary to direct the current or to prevent the silting-up of the recesses by the abutments; but this would be land gained from the shoals, and the river would be in every respect improved.

The advantage to be gained by diminishing the points of support in carrying a bridge over a navigable river or other water should not, however, be purchased by the total submersion, under any circumstances, of those which may remain, and the immersion of the springings of the arches, if it be but for the sake of effect; for under no circumstances do the magnificent structures last spoken of present so unpleasing an appearance as at high tides, and especially at flood springs, when the very bad result alluded to occurs. This, which is merely unsightly in such a river as the Thames, whose course is even, and whose affluents are mild and gentle, becomes a positive defect of a serious nature where a river is fed by torrents, and where floods may increase the waters of a river more rapidly than its channel will

allow of their discharge. It must be supposed, in any case, that the piers of a bridge have been adapted to the section of the river, so as not to impede the current in any greater degree than it may be liable to be impeded by existing natural causes. While the water meets with no obstruction greater than that which the piers offer, no appreciable head should be formed, nor should any severe action take place upon the bottom of the river; but if the springings of the arches be once immersed, the haunches and spandrels add, inch by inch, and foot by foot, to the line of obstruction as the water attains a head, which it does with constantly increasing effect;—the bed of the river is acted upon, and the piers are undermined to the utter destruction of the whole edifice; or, if the piers are so founded and formed as to withstand this action, and the bridge is not overturned bodily, it forms a thorough dam, and is productive of more mischief than would be occasioned by its fall, by holding the water up to flood the surrounding country, and thus to mingle the inhabitants and their habitations in one common ruin.

The gallant Welshman who triumphed over the Taaf after two defeats, suffered the first and most severe overthrow from making his bridge in effect a merely perforated dam, which the rubbish brought down by the torrent, on being intercepted by the thick piers of the narrow arches, closed altogether, and produced the overthrow of the work; whilst Smeaton's bridge over the Tyne at Hexham was undermined and thrown down through the defect before indicated occasioning the water to form a head,

which gave increased effect to the stream upon the bed of the river, when the ground was swept from under the piers, which fell, and necessarily took the superstructure with them. What had been twice before attempted ineffectually, and what Smeaton failed in doing successfully, has been since done, and in an effectual manner, by a person of no extraordinary pretensions, but who avoided the defect that had been overlooked by his more eminent, but, in this case, less successful predecessor.

In the case of tidal rivers like the Thames where the rise and fall are moderate, and where very little difference is at any time occasioned in the height of the water by floods, but where the high and low lines of water are constantly recurring, no apology can be admitted for the immersion of the springing stones of the arches, especially when the bridge occurs in the heart of a town, as in London, nor should the bridge have a stilted and consequently a mean appearance at the lowest condition of the water.

Where the tide rises and falls a great height, as in the Wye at Chepstow, it seems almost impossible to form a bridge upon arches that can have a sightly and agreeable appearance at both high and low water. In such a case other expedients for carrying a bridge than arching under the road-way may be recurred to with advantage, and some are capable of being adopted even in such a case with good effect. Telford's stone bridge over the river Dee at Tongueland, near the town of Kirkcudbright, where the tide rises about 20 feet, springs from the level

of half tide, and the consequence is, that the great segmental arch of which it is composed is always, in a greater or less degree, unsightly.

It is, perhaps, unnecessary to provide in the design for the appearance of a bridge under the extraordinary circumstances of an unusual flood like that which threw down Smeaton's bridge at Hexham, when the safety of the bridge and the discharge of the waters are provided for;—the design being of proper character under the general and every-day aspect of the work. Numberless examples may be cited, however, of bridges, across flooding rivers, with arches springing from the level of lowest summer water, and whose springings, consequently, are almost always immersed, whilst occasional floods may be said absolutely to drown them. Gwynn's bridge over the Severn at Shrewsbury is one of this kind, and most of Telford's early stone bridges over the same river are open to objection on the same account, though in his later works Telford has endeavoured to avoid it in appearance, if not in reality, by adopting the French mode of masking an elliptical arch, sprung low down, by a flat segment of a circle, having the radius of the flattest part of the ellipse, sprung from, or rather abutting upon, a point high up on the piers.

Perronet's flat arches without the inner ellipse, as in the bridge of St. Maixence, and in his bridge over the Seine at Paris, hardly possess that degree of elegance which must be considered essential in works of pretence, such as those in which the distinguished engineer-architect referred to, employed them, though he himself

intimates that his intention was to produce variety of design, and to avoid the constant repetition of the same forms and arrangements. Want of sufficient power of resistance in the abutments has occasioned partial failure in most, and total failure in some, of the bridges that have been attempted in masonry with the very flat segmental arch alluded to; and many such have been erected in France upon some of its torrent-like rivers, and their torrent tributaries. As far as stability is concerned, Perronet was more successful than his contemporaries were with this class of bridge construction; but, for beauty of design, his bridges on the Oise at Pont St. Maixence, and on the Loing at Nemours, are but little to be preferred to the bridge over the Oignon at Pesmes, which is stated to have been the first built in France, (whose arches being segments of a circle have their springings at the highest or flood-water level,) or to the bridges Fouchards on the Thouet near Saumur, d'Homps upon the Aude in Languedoc, or the bridge of St. Diez upon the Meurthe above Lunêville. The rise of the arches in the instances cited does not exceed one-tenth the span in any case; in some of these examples it is but one-twelfth, and in Perronet's bridge at Nemours the rise of the arches is but little more than one-fifteenth their span. All have, in a greater or less degree, an awkward and stilted, if not a mean, appearance, which indeed is hardly avoided by the addition of lumbering architectural and sculptural accessories in Perronet's Pont de la Concorde, or bridge of Louis the Sixteenth, over the Seine at Paris, of which the rise of the flat segmental

arches ranges from one-twelfth in the smallest arches to less than one-tenth the span in the central and largest arch.

But the defect here alluded to cannot have appeared to be such to the eminent author of the Pont de Neuilly, or he would not have masked the more elegant elliptical inner arch, which he employed in that work, by a flat segment abutting upon the piers, which, under their springings, look weak, but which, slight as they are if tested by the ordinary practice of his time, have mass enough where they receive the elliptical portions of the arches.

It may be remarked here, in passing, that the work last mentioned affords a very uncommon instance of too much space being given by the bridge to the water-course, through the removal, in a somewhat injudicious manner, of the island that formerly divided the bed of the river into two channels at its site, by which the river is rendered more sluggish than its wont; and, in consequence of this and of the altered direction of the currents from and into the divided channels above and below, new and inconvenient alluvial deposits have taken place.

More recent practice has shown, however, that the use of a flat segmental arch is not inconsistent with a much nearer approach to excellence than Perronet and his contemporaries attained, though, indeed, the instances to be cited do not show so reduced a proportion of the rise to the span, or, in other words, arches so very flat as those above referred to. In the bridge over the Seine, opposite to the Military School,—L'École Militaire,—(known as the Pont du Champ de Mars, and formerly as the Pont d'Jéna), Paris possesses a very striking con-

trast to the Pont de la Concorde. There is but little material difference in their extent and in the span and rise of the arches, or in the proportion of the piers to the openings, of these two bridges; and although the one is of five unequal, and the other of the same number of equal arches, it is not in this circumstance, nor in the consequent level line which its horizontal surface presents, that the great superiority in appearance of the former bridge consists, but in the absence of all attempt at adventitious aids to produce the pleasing effect which the simple and unaffected arrangement of its parts produces; — in the absence, indeed, of the pseudo-architectural accessories of columns and balustrades which deform Perronet's design.

A similar, and in some respects superior, example of a bridge with flat arches of the proportion of those of the two Parisian examples alluded to, and of about the date of the bridge of the Champ de Mars, is the bridge over the Thames at Staines, a work of the late Mr. Rennie. This is of three unequal arches, and being, like the bridge of the Champ de Mars, almost entirely free from architectural affectations, the composition produces a striking architectural effect. Staines Bridge may be considered superior to the Parisian Bridge in the land arches springing from their own piers clear of the projecting faces of the abutments, whereas, in the latter, the springings of the land arches are buried in the abutments;—in the more appropriate as well as fitter form of the piers or cut-waters, and generally in the projection of massive and well composed abutment constructions, which add materially to its effect, whilst the bridge of

the Champ de Mars appears to be squeezed down between the retaining walls of the quays. The cornice and blocked parapets of both are of fine character, and both examples show clearly that—although the very flat arch affected by the French school of bridge architects of the latter half of the last century may, in some of the works above referred to, be too flat to produce an agreeable effect in composition,—a near approach to the same degree of flatness is not inconsistent with both dignity and elegance, whilst the advantage of the flat arch in many respects cannot be questioned.

When circumstances will admit of a very flat arch from bank to bank of a torrent, there need be no limit to its extent within the power of resistance of the stone employed to the pressure, and of the natural abutments to the thrust. In mountain-passes it will frequently happen that such arches may be applied with advantage; but often, in such cases, permanently constructed bridges are avoided by the authorities of the State in possession, because of the difficulty of destroying them quickly, so as to render the road upon which they may be placed, impracticable to an enemy in case of necessity. Instances of this kind occur in the route of the Simplon over the Alps; massive piers having been built from the bottom of deep ravines to carry wooden platforms where stone arches might have been thrown across at less expense, free from all liability to injury from the torrent, to which the piers are constantly exposed, and from which they constantly suffer.

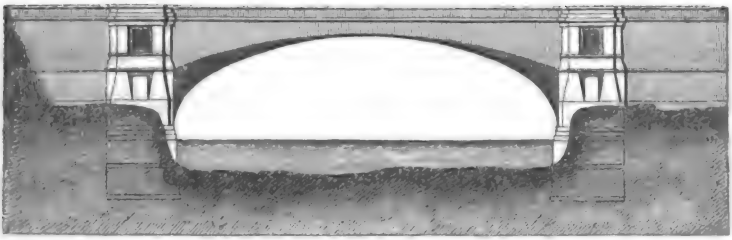
Deep cuttings in stone or in chalk, for roads, railways, or canals, afford occasions for the use of the flat seg-

mental arch with appropriateness and economy, though their application in such cases would be for a different purpose from that hitherto contemplated, this being to obtain or retain head-way in comparatively shallow cuttings, and to avoid expense in carrying up lofty abutment and bearing walls in deep cuttings, whereas way for flood-waters has been the object sought to be attained in most of the cases in which the very flat segmental arch has been used upon rivers and upon their tributary torrents.

Perronet designed a bridge of one flat arch, of 150 French, or 160 English feet in span, (see Plate 89,) to be erected in duplicate over the Seine at Melun, where the river is divided by an island into two nearly equal arms, requiring two bridges of about the same size. The old bridges, which these were intended to replace, were of eight arches each, and the single span proposed embraced about the same space that the whole number of openings collected, of the old bridge, gave to the waters. Here the abutments were to be provided by art, and necessarily of great extent and solidity, because of the flatness of the arch to be erected. Notwithstanding this necessity, the architect was not contented to allow the arch to appear to be of even the slight degree of convexity that a radius of 200 would give to an arc upon a chord of 150, but he designed the archivolt line of headers to be a segment of a circle whose radius should be 300, the chord of the arc of this upper and outer arch being 162 French, or nearly 173 English feet, whilst the versed sine of the former was one-tenth the span of the bridge or chord of the arc, and that of the latter but one-fifteenth part of its chord.

The lowness of the banks of the river, and the consequent want of height for the proposed archway above the ordinary water-line somewhat proportionable to the span or breadth of the opening, and the ungainly appearance of the two heterogeneous curves of the arch, must have prevented these bridges, if they had been built, from being more than merely striking from their singularity in extent and proportion, though indeed the abutments are well composed, and, moreover, the useful purposes of a bridge would have been perfectly attained in each case. The almost necessary contraction of the water-way is made in the least injurious manner,—piers in the water-way being altogether avoided,—and the height of the single opening is sufficient to allow of the free navigation of the river when its navigation is otherwise practicable, and of the passage of ordinary flood-water; whilst the escape of the water at extraordinary floods is provided for by land-arches calculated to give space equal to what might be abstracted by the immersion of the springings and spandrels of the great arch, by which means not only would the safety of the structure have been secured, but any extraordinary action upon the bed of the river by restraint of the water would have been avoided; the ordinary effect of the slightly narrowed course of the water being rather beneficial than otherwise to the stream, for the purposes of navigation. The road-way provided was ample and of easy acclivity, so that the sister bridges of Melun would have been, in all essentials, as perfect as the circumstances of their locality would permit.

GLOUCESTER OVER-BRIDGE.



Mr. Telford has availed himself of Perronet's design for the bridges at Melun as a study for his bridge over the main arm of the Severn at Gloucester, though he seems to have had the impression upon his mind that he was merely adopting the peculiarity, which appears to have originated with Perronet, of making the general body of the arch an ellipse, and the archivolts, or external arch-stones of the faces, a segment of a circle, having the same chord as, but with little more than one-third the rise of, the semi-ellipsis, whilst the same horizontal line is tangent to both parts of the arch where they are coincident, which is throughout the flattest part of the approximate ellipse.

The arches of the bridge at Neuilly, in which Perronet first exhibited this peculiarity, are but a fraction less than 128 English feet in span; and the radius of the circle, of which the coincident portions of the outer segmental arch are parts, is only 150 feet French, or 160 English feet; whilst Telford has applied, in the Gloucester Over-bridge, with a span of 150 feet, an ellipse whose flattest part, coincident with the outer segmental archivolts, is an arc of a circle whose radius is 220 feet, or very nearly identical with an average of the two parts of the arch of the bridge designed by Perronet for Melun,

when proportioned to Telford's diminished span. Moreover, at Neuilly the springings of the elliptical part of the arch are but a trifling degree removed within the outer face or elevation of the bridge, giving it, in effect, the whole value of the elliptical form throughout its breadth; whereas at Gloucester the ellipse springs from a transverse base not exceeding one-half the whole width of the bridge; so that, in a general view from a moderately elevated position, the elliptical portion of the arch will scarcely be seen, and certainly not in a greater degree than the lower and more nearly coincident inner curve of the arch of the proposed bridge for Melun. In this particular, Telford's work possesses a decided advantage over that from which the peculiar forms of the arch were derived, whilst it has the advantage over that which appears to be more nearly its prototype, in the greater height of the soffit of the bridge at ordinary low water in proportion to the span, and also in the more elegant form of the inner arch when it comes into view; though Gloucester Over-bridge is deficient when compared with Perronet's design for the Melun bridges in the means of escape for the high flood-waters, as well as in the architectonic character of the faces of the abutments.

Both Perronet and Telford appear to have attached importance and value to the result of the complex form of the Neuilly arch that can hardly be conceded to it. Perronet says, "This arrangement [des espèces de cornes de vache en voussures] facilitates the introduction of the water, and gives much more lightness and boldness of effect to the bridge;"[6] and Telford remarks, in speaking

[6] Ouvres de M. Perronet, p. 3, 4to. Paris, 1788.

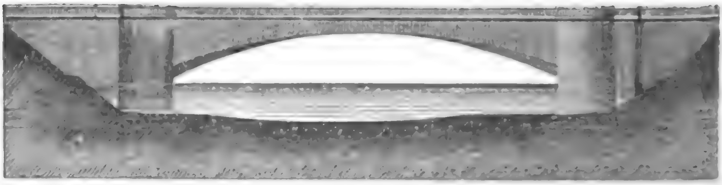
of his design for the Gloucester Over-bridge, "This complex form converts each side of the vault of the arch into the shape of the entrance of a pipe, to suit the contracted passage of the fluid, thus lessening the flat surface opposed to the current of the river whenever the tide rises above the springing or middle of the ellipse."[7] It is not so clear, however, that any advantage would be obtained from the "complex form," notwithstanding the concurring opinions of these two eminent men; but it is quite certain that no greater surface, whether flat or otherwise, should be opposed to the current of a river when the waters rise above the springings of any of the arches, than when it is, as it ought always to be, below that line; that is to say, the springings of the arches should always be placed above such level: for, without entering into a discussion of the combined hydraulic and hydrostatic problem involved in the question, it may be confidently asserted that no drop of water the more will escape through the opening of any particular archway because it is wider at the ends than in the middle, and that a bridge will be not the less liable to be undermined or overthrown by a certain body of water in motion upon it, because it presents a larger concave than a smaller flat surface, though indeed it may be more liable to have the crown of its arch or arches blown up thereby, because of the larger surface of the soffit exposed to the pressure of the head of water in extreme cases.

The elliptical arch of the general body of Gloucester Over-bridge is sprung from a level between four and five feet below that to which ordinary spring-tides rise,

[7] Life of Telford, p. 261, 4to. London, 1838.

whilst upland flood-waters not unfrequently immerse the haunches of the ellipse to within a few feet of the springing of the outer segmental arch of the faces, and thus contract the water-way to such an extent, that relieving side-arches should certainly have been included in the design, of capacity equal, at the least, to the space liable to be abstracted from the great archway.

BRIDGE OVER THE DORA AT TURIN.

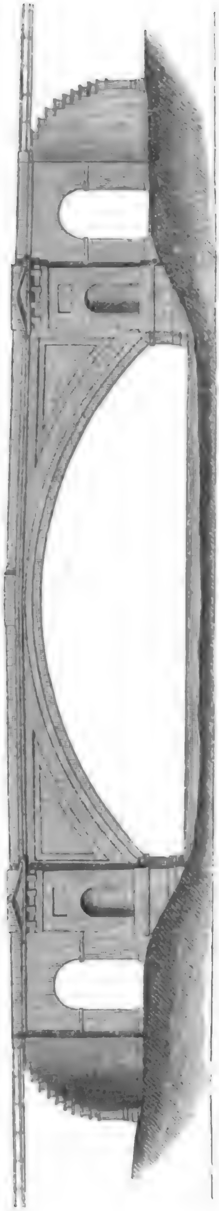


Another recent adaptation, not to say appropriation, of Perronet's Melun Bridge, is to be found within the suburbs of Turin, over the river Dora Riparia, a confluent of the Po, in the line of the main road from France, by way of Piedmont into Italy. This bridge is of one arch, a segment of a circle, whose chord is nearly 148 English feet, and the versed sine a fraction above 18 feet, making the radius of the circle about 160 feet. The outer and upper arch of the faces of this bridge is in the same proportion flatter than the inner and lower, of which the chord is given, as in Perronet's design; but there is no provision for relief by side-arches in the event of high floods, to which, however, the Dora is very subject; and the abutments are thrown forward in quadrants up to the springings of the arch, in such manner as to distinguish the work most effectually from Perronet's admirable composition of that feature of his design. Nevertheless, this Italian Bridge appears to be

of the flattest arch yet constructed of masonry upon so great a span.

It is impossible to contemplate Grosvenor Bridge over the Dee at Chester without regretting that the great span of its single arch (200 feet) had not been either so much flatter as to raise the springings out of the reach of flood-waters,—as the situation of the bridge does not render the height of the road-way objectionable,—or in the more pleasing form of an ellipse, rising from its present springing level: or indeed it might with greater advantage have taken the peculiar form of that singularly graceful variety of the ellipse which is found in the arches of the bridge of the Most Holy Trinity (Ponte della Santissima Trinità) over the Arno at Florence. The rise from the springing level to that of the crown in this example is but little more than one-sixth the span, whilst the rise of Grosvenor Bridge arch is but in the same proportion less than one-fifth its span; so that the form of the arch of the Florentine Bridge might have been obtained in that at Chester, with a much higher line of springing.

The forms and proportions of the work last cited have been less studied than their intrinsic merits deserve. Upon a river which does not admit of navigation, but which is liable to be swollen by floods of upland-waters, the head-way is sufficiently good, for indeed the springings of the arches are out of the reach of the highest floods to which the Arno is subjected,—whilst the road-way over is of gentle acclivity, though leading from but moderately raised approaches. The piers, however, are unusually and unnecessarily massive; but the general composition of the work deserves the character Perronet claimed for

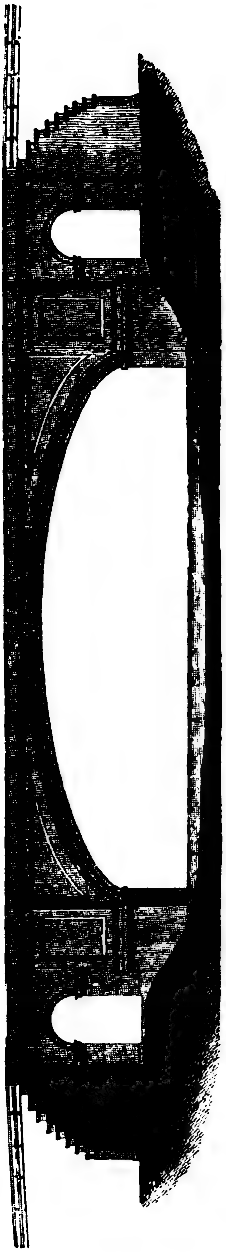


CHESTER BRIDGE.

his own bridge at Neuilly,[8] of lightness and boldness, notwithstanding the massiveness of the piers; and it is not alone in the form and rise of the arch that Ammanati's bridge over the Arno excels Harrison's gigantic stride at Chester; the freedom, boldness, and even dignity of character in the superficial arrangements of the Florentine give the other bridge, as well indeed as most other bridges making pretensions to architectural adornment,—in a comparison of architectural merits,—an undignified, if not absolutely a mean, appearance.

Nevertheless, the spanning of a water-way from bank to bank, with a sufficient and convenient road over, without placing obstructions within the channel, is so important when effected, and therefore so desirable to effect, that many, if not most, other considerations fall into the shade in contemplating works that do secure the passage of a river

[8] It is, perhaps, hardly fair to say that Perronet *claimed* this character for his work; the expression is simply to the effect that the combination between the two forms of the inner and outer arches by the means referred to at p. 40, *ante*, "donne beaucoup plus de légèreté et de hardiesse au pont."



CHESTER BRIDGE, WITH THE ARCH OF THE BRIDGE OF THE TRINITÀ AT FLORENCE.

without placing impediments in the way of its navigation where the river is navigable, and of the free course of its waters where an accumulation of water against the upper face of a bridge may be dangerous to the bridge itself, or to the neighbouring country. But ugliness is not an essential to utility, and it must not be concluded that every thing else is to be excused if the main object be attained; yet in too many existing examples, — and some of those above referred to are here intended,—the worst qualities of composition are forced upon a work by want of taste and of artistic skill in the designer.

The usual *materiæ architectonicæ* are entirely out of place, and out of character, in bridge compositions. Columns and approximations to columnar forms and proportions, pilasters, entablatures, niches, battlements, balustrades, towers and turrets, pinnacles and pediments, are gauds and devices, in the application of which to bridge composition the most eminent engineer-architects have failed

to produce any thing but meanness or absurdity, or a combination of both. Telford's most pretending works are spoilt by an affectation of architectural disposition and adornments, whilst Rennie's coupled columns to Waterloo Bridge are only not quite so absurd as Mylne's to Blackfriars', or Perronet's columns of Pæstan proportions, (according to the notion of their author,) in the Pont de la Concorde at Paris. A fitting and graceful combination of the leading lines of such a work as a bridge upon a moderately large scale, and of which the characteristics should be boldness and simplicity, can hardly fail to produce a grand and striking effect, if the end the work is proposed to answer has been in the first place fully and effectually provided for; for,—it cannot be too often repeated,—the unrestrained passage of the waters, the convenience of the navigation, and the requirements of the road, together with the permanent security of the structure itself, are essentials to excellence in the composition of a bridge, whose omission or imperfect production will never be excused by the presence of any merely adventitious merits.

The tendency of the foregoing observations being to urge the advantage of effecting as much as possible in bridge constructions in one reach, or by spanning a water-way from abutment to abutment, or from bank to bank, without intervening piers, it must not be concluded, therefore, that materials which admit of longer reaches than can be effected under ordinary circumstances with masonry are to be preferred to stone. No approach has yet been made in any bridge hitherto constructed with granite to the pressure with which that noble, and

it may be almost said recently discovered, material may be prudently trusted. Three quarters of a century ago Mylne's proposal to build an elliptical arch of 100 feet span, with the strong free-stone from the Portland quarries, excited amazement, and the determination to execute it produced alarm, though the proposed arch so nearly approached a semicircle that the rise is more than *two-fifths* of the span; but within the current decennary two elliptical arches have been built of brick over the same river upon which Mylne's temerity was proposed to be displayed with Portland stone, the brick arches being of 128 feet span, with a rise of 24 feet 3 inches, or somewhat less than *one-fifth* their span. It is true, indeed, that the proposal to build such arches as these last, of such proportions, and with such a material, induced many prophetic warnings of a disastrous result; but the heavy trains, and the heavier engines of the Great Western Railway, run without hesitation, and—it may perhaps, after two years' experience without injury or accident, be deemed to be—without danger, over the Maidenhead Railway Bridge, though its arches possess no greater power of resistance to their own thrust, and to the weight and vibrations induced by the loads passing over them, than the mortar composed of Roman cement and sand in which the bricks are set possesses; and that species of mortar never acquires the same power of resisting pressure that good stock bricks possess. If, then, arches may be built of the proportion last specified, with materials so comparatively weak, or liable to be crushed by pressure or fractured by vibration, as bricks and cement, what may not be done with a substance like

granite, the average specimens of which possess a power of resistance to pressure four or five times greater than that possessed by the best stock bricks, and seven or eight times greater than the best indurated cement possesses?

The bridge that formerly stood over the Adda at Trezzo, in the Milanese, (see Frontispiece to vol. I.,) is an instance of what may be done with granite beyond what has been elsewhere attempted; for, indeed, the power which the use of this material gives to the bridge-builder seems rather to have been forgotten than not to have been discovered. The bridge referred to exhibited an arch of 251 feet in span, its form being a segment of a circle, whose radius is nearly 134 feet; the rise of the arch from the line of springing, or the versed sine of the segment, having exceeded but in a trifling degree one-third the chord, whilst the depth of the arch-stones was barely *one-sixtieth* of the same dimension. Let it be remembered that the depth of arch-stones, (or length measured upon the line of the radius of the arc,) in large bridge-arches has been rarely allowed to exceed one-thirtieth the span, and more frequently to range between a fifteenth, as in Perronet's bridge at Orleans, and Labelye's at Westminster, and a twentieth or twenty-fourth, as at Neuilly and Blackfriars; though, indeed, instances are found in which the proportion is as low as one-tenth, as at Nemours, and as high as a thirty-fourth, as at Tongueland. An example exists in North Wales in a work by the eminent architect Inigo Jones, of an arch of which the proportion in length of the arch-stones to the span of the arch is but a fraction above a fortieth; and in

the better known and more striking example afforded by William Edwards's bridge (Pont-y-pryd) over the Taaf, in South Wales, the similar proportion of the depth, or length, of the key-stone to the span of the arch is only one-forty-seventh. A collation of these examples with that of the bridge at Trezzo will show that there is reason drawn from actual experience for the suggestion that bridges may be built of granite and other hard stone in single arches, or in series of arches, of greater span and with less rise in proportion to the span than has been hitherto practised or even attempted.

It is not to be overlooked, however, that economical considerations may stand seriously in the way of such a recommendation as that here developed; but it is also deserving of consideration whether bridges of masonry are not built far more expensively than they need be. A large proportion of the cost of masonry, and of granite masonry especially, is in labour, and no unimportant part of the whole of the labour expended is generally devoted to the high finish of exterior surfaces, and in producing sunk or rounded, or otherwise moulded, forms; the effect of all which is for the most part injurious rather than beneficial to the appearance of such works; fine finish upon them resulting in tameness, and varied forms in deformity. In towns, as in London, and upon rivers which, like the Thames, are highways, and exhibit the face elevations of bridges to comparatively near inspection, highly finished surfaces and moulded forms may be considered desirable, if they are desirable under any circumstances; but it may be fairly

questioned whether both Waterloo and London Bridges would not have been finer objects had the masonry of their external faces been merely rough-axed, or even left scabbled, instead of being fair hammer-dressed; and certainly many thousands of pounds might have been saved in the execution of the former work, and a much better result produced, by the omission of the coupled columns and their immediate accessories, and by the use of a plain parapet of a more reasonable height, in the stead of the high, the enormously expensive, and absurdly ugly balustraded enclosures which now aid the columns and their projected entablatures to deform that splendid structure.

A great deal of labour is often applied, and consequently expense incurred, in producing upon masonry faces a regularly rough, or what is called rock or rustic, surface, to yield a combination of boldness and richness; and in finished and enriched architectural works, which the eye may scan closely, and where a certain degree of harmony should be preserved between the roughly appearing and the finely wrought and carved parts, regularity in the roughness may be worth attending to and obtaining at some cost; but in the faces of a bridge the rough or rock surface is most appropriate, the ordinary magnitude of the parts countenancing, if not absolutely requiring, the boldness, and, indeed, richness of surface which roughness gives to massive masonry; whilst the removal of the faces from the eye, through the necessity of viewing a bridge as an object at a sufficient distance to annihilate the effect upon the eye of regularity or irregularity in the roughness, renders it quite unnecessary to be at charges to make the roughness regular.

But the scabbled face with which blocks of granite are sent out of the quarry,—the labour in producing which, being necessary to give the required general forms and proportions to the blocks, is necessarily included in the price of the stone,—is alone sufficient dressing for all useful purposes, as far as the exterior faces of bridge constructions are concerned; so that the labour of dressing may be confined to the beds and joints of the stones. When it is remembered that in granite masonry the labour to the fair hammer-dressing generally given to exterior surfaces costs, according to the degree of fineness, from twice to three times as much as that to the beds and joints, it will be understood to what an extent bridge constructions in granite may be economized. The same is true of other stone, though not to the same extent, unless the stone be as hard as granite; but the labour upon beds and joints is always estimated at less than plain work upon fair faces, let the stone be what it may: and again, the avoidance of hollowed and rounded or otherwise moulded surfaces, as in columns and balusters, will tend still further to lessen the cost, as compared with the ordinary expenditure upon such matters.

It may be further urged, in an economical point of view, in favour of single arches of masonry whenever they are attainable, in preference to a series of smaller arches involving piers in a water-way, and generally of arches of great span to reduce the number of piers where a plurality of arches is unavoidable, that abutments may be strengthened to bear the weight and thrust of a large flat arch at much less expense than piers within the bed of a river involve; and, in like manner, that a small

number of piers may be made strong enough to carry arches of great span at far less cost than a greater number of slighter piers would impose, since the addition of substance to either abutments or piers adds but little if at all to the essential, or rather unavoidable, expense of founding them on the margins of, or in a water-way, whilst the founding and building of piers form, for the most part, the most costly item in the estimate of a bridge.

Attention has been already called to the reasons why a navigable river may be narrowed from the banks by projecting the abutments of a bridge in upon a stream with less disadvantage than by piers within the water-way, even if the former mode do not in some cases produce an absolute benefit to the river by tending to keep its channel clear where it is most desirable that it should be kept so. But this has its limits which must be carefully ascertained and guarded, or the current will be forced to scour the bed under the bridge, the matter from which will deposit and form a shoal immediately below it, unless the banks of the river are also brought together by embankment or otherwise; and, moroever, the current may be made inconveniently rapid for navigation by undue contraction of the breadth of the water-way, independently of the other contingencies, which, however, will be modified in their effects by the nature and degree of hardness of the substances that form the bottom or bed of the river.

There have been occasions indeed when it has appeared questionable whether a bridge has not best performed its duty by penning up the waters of a river for

the benefit of the navigation above it, as in the case of Old London Bridge; and occasions do occur for requiring a bridge to perform the duty of a dam against tidal or flood-waters, rendering the frequent recurrence of piers thus desirable, if not essential to the desired effects, though, in most cases, whatever regulation a water-way may require can be made consistent with a single opening, and consequently with the smallest possible number of openings where a bridge of one arch is impracticable or unattainable.

The history of Old London Bridge yields a singularly instructive instance of difficulties occasioned by the bridge itself, of interposed obstructions arising out of such difficulties, and of the effects produced by the measures adopted for obviating the difficulties without removing the obstructions. The mischief done by widening the central archway, and the precautions taken to hold up and back, and to regulate the water, as well for the advantage of above-bridge navigation as for the service of the water-works which had grown parasitically about the old bridge, deriving their support from the dangerous rapid the bridge had made, making the danger greater by their existence, and claiming a perpetuation of the nuisance that had given them birth to sustain them in being, form altogether a most valuable series of practical lessons, which cannot be studied too closely or learned too well. They may be found in Smeaton's Reports and in the Reports of a Committee of the House of Commons on the Improvement of the Port of London, printed in 1799, 1800, and 1801.

Although the precautionary works in connexion with

the ordinary repairs of London Bridge cost the City at the rate of more than four thousand pounds a year, and the general community property to an unlimited extent by the capsizing and swamping of craft, without taking the constant delay of what passed safely into account, together with the lives of thirty or forty persons annually, so far from the expense, loss, and danger having induced any strong recommendation on the part of Mr. Smeaton, whose name would have given authority to any such, to raise or obtain a supply of water by other means than those which existed only by rendering the river dangerous, and the bridge itself insecure, the sole aim of that eminent engineer seems to have been how to raise the same head of water that the water-works had had the use of before the alteration of the central arch, without detriment to the then existing facilities for the navigation of the river under the bridge. He states it, indeed, as a thing "to be wished, for the sake of security to the bridge, as well as navigation, that some equivalent could be formed to the water-works, so that all the arches might be unstopped; the great arch remaining as it is," [Report, dated 28th July, 1766,] but there he stops. In another place, indeed, Smeaton states it to be his opinion that "were the fall at the bridge considerably reduced *by any means whatever*, the navigation of that part of the river [the part above bridge] would be *materially* affected;" he speculates, too, upon the probability of the bridge having been so constructed,—that is, as a merely perforated dam or imperfect weir,—as "an expedient to retain more water in the river at low-water," —"for the sake of navigation;" and he concludes as a

matter of "more than speculation," "that a stoppage at London Bridge, in the present state of the bed of the river above bridge, is necessary to the present navigation thereof," this being, however, upon a foregone conclusion that "if London Bridge were to be taken away, the river would become so shallow above bridge at low-water, that the navigation would be greatly impeded for hours each tide." It seems to have escaped the notice of this generally accurate observer, that the then existing state of the bridge did actually stop the navigation of the river one way or the other constantly.[9]

Mr. Smeaton advised the Corporation to stop up some of the smaller openings of the bridge, to make the then lately widened central archway narrower, though but slightly, and to raise the bed of the river within that opening to form a sort of sunk weir, the effect of which would be to make a difference in the level of from four to six feet between the low-water level on one side and on the other side of the bridge, or a fall of that depth within a length of thirty or forty yards! But little more than half a century had elapsed from the time that produced these opinions and recommendations when the water-works and bridge had disappeared together, and it has been found that any alteration that may have been occasioned

[9] See Mr. Mylne's Report "on the state of the River Thames and its bed; on the structure of London Bridge, and as to the navigation of the river above and below it, &c., &c.," in the Appendix to the Report of the Select Committee on the Improvement of the Port of London, ordered by the House of Commons to be printed July 28, 1800. "During the seven hours of ebb passing through it (the bridge) nothing can pass upwards;" . . . . "the same exists with respect to the duration of flood, for five hours, when no person can get through it against the stream.

thereby in the depth of the water above bridge has been much more than compensated by the removal of the obstructions which Smeaton considered to be necessary "for the sake of the navigation;" nor, indeed, does the result justify his conclusion when a fair comparison is drawn, that "if London [Old] Bridge were to be taken away, the river would become so shallow above bridge at low-water that the navigation would be greatly impeded for hours each tide." It is true, nevertheless, that the water does become shallower above bridge than it became before the removal of the old bridge; and it would appear desirable that some expedient should be adopted (and various modes have been suggested) to regulate the discharge, and to impede the afflux at flood springs, when the river is full from upland waters, without occasioning rapids or injuriously affecting the navigation in attempting to improve it.[10]

The widening of the central opening of Old London Bridge by the removal of the chapel-pier and starling which stood between the arches included in the new archway, affords a striking practical warning of the danger to a bridge itself, and of the injury that may be done to a river by partial interference with a continuous obstruction. Before the widening referred to there was no very material difference between the numerous perforations or archways of the bridge, so that

[10] There can be no doubt but that the navigation of the Thames through London would be greatly improved, and the health of the town benefited, by embanking the river on both sides from Nine Elms to London Bridge, in such manner as to make its bed of nearly equal breadth throughout.

they operated like so many equal sluices distributed over the whole width of the stream, and no particular action was induced upon any one that was not felt to nearly the same extent in all. Throwing every two or three openings into one, reducing the number by one-half or one-third, and thus extending the water-way equally along the whole line, could hardly have produced increased action upon any one of such widened openings more than upon all the rest, though it seems pretty clear that the action of the water would, by the extension of the water-way and the consequent reduction of the head formed by the partial dam, be less than that induced by the pressure of the head while the stream was pent up by the bridge in its unopened condition. Any extension of the water-way under the bridge should, therefore,—having reference to the safety of the bridge and the equability of the water-course for the purposes of navigation,—have been obtained by an equal distribution of the added space along the whole line; or, as that would have been both inconvenient and expensive, and the interests of the water-works had in this particular instance to be regarded, it might have been confined to a limited number of the openings in the central part of the bridge, as an approximation to the really correct course. Instead of this the course most removed from that here assumed to be the correct one was pursued. An opening of great comparative magnitude was made in the middle of the bridge by the removal of the central pier that had carried the chapel, throwing the space it had occupied with that of the two arches which rested upon it, into one large arch;

and, as a compensation to the water-works, some of the smaller openings between that and the abutments were shut up. Within five years after this was completed, it was found that the starlings under the piers, upon which the great arch rested, had become insecure, and, on an examination by Mr. Smeaton, he found "the current making hourly depredation upon the starlings, the south-west shoulder of the north pier undermined six feet, and the original piles upon which the old works had been built laid bare to the action of the water, and several of them loosened. In this perilous state, (the Report continues,) when a settlement of that pier must necessarily have taken place in a few days, I proposed the only remedy I knew of that was likely to be attended with success in circumstances so pressing, viz.: that of securing the bed of the river with a body of rubble stone, upon which the said angle was underpinned." The emergency was indeed thought so great that the work was begun on a Sunday morning, the stone of the old city gates then lately sold and removed, and lying opportunely ready in Moorfields, being at once re-purchased to effect the operation with.

The cause of the mischief done and of the danger threatened is easily determined. The body of water in motion under the action of the head formed by the partial dam across the rest of the river, necessarily scoured the channel and sought to do so to a depth commensurate with its breadth, as compared with the breadth of the whole body of upward water as far as it was unrelieved by the smaller existing openings of the bridge. So powerful was the action thus induced that the usual

depth of the river at low-water at that part had been more than doubled under the widened archway, and this was, at Smeaton's recommendation, filled up with rubble, and the rubble further raised so as to form what has been called a sunk weir; but this was done without sufficient reference, it would appear, to the effect that would be produced upon the bottom of the river on either side of this weir by the fall of the water both above and below, (see Plate 19,) the tidal current being reciprocal. Within three years after the rubble had been deposited under the archway, the same engineer found the bed of the river, at a few yards only above the rubble weir, to be pooled to the depth of nearly twenty feet below the average bottom of the river, and at a proportionably greater distance below the bridge the bed was pooled five feet deeper than it was above, or to an extent that might have been calculated to be due to the greater head and consequent fall arising from the upland waters, and the difference in the times of the ebb and flow: "In consequence of which," says Smeaton, "the bed of the river from the points of the starlings upward and downward, forming a slope too great [steep] for the rubble to lie upon when impelled by so strong a current, the sides of the rubble will naturally slide into the cavities: this, in consequence, has impoverished the body of rubble immediately under the arch." Mr. Smeaton recommended, therefore, that rubble should be dropped into the pools also, to lessen the depths and to make a firm footing for the rubble that lay immediately contiguous to the piers and starlings, the deficiency under the arch itself being also made up to

answer the original intention of such filling. Besides these operations, Smeaton considered " it would be necessary that the condition of the rubble bed should be frequently examined, and that there should be always in readiness a quantity of rubble to supply such deficiencies as from time to time might happen." This being duly attended to, he thought there was " sufficient reason to suppose that it would prove a lasting support," and that, " when by the disposition of the rubble the cavities above and below were hindered from pooling, and the foot of the rubble bed within the starlings supported thereby, and the whole by time consolidated, it might be expected that the repairs of that part would be very inconsiderable." The result did not, however, justify the expectations which were thus raised, for the rubble was still carried away from under the archway to form shoals at some short distance below the bridge, where the effect of the rapid ceased to keep it in onward motion. After patient perseverance in feeding the archway and the pools with rubble for nearly thirty years, a further attempt was made to detain the rubble under the archway by forcing down transverse beams in grooves within the sides of the starlings, which were expected " to preserve the form of the bottom by assisting to retain the rubble." Of nine beams thus placed in 1793 and 1794, but two remained in 1800, and these two were the upwardmost and the next to it but one, or those the least exposed to the active force of the water, and the least available to the intended effect, so that, of course, all restraint upon the rubble was again removed.

In the Parliamentary inquiry in 1799 and 1800, upon

the condition, with reference to the improvement, of the port of London, and which resulted in the establishment of the London and other Docks upon the Thames, near London, the condition of London Bridge and its effects upon the navigation of the river became also a subject of inquiry, and various plans were submitted to the Select Committee of the House of Commons appointed to inquire, by several architect-engineers and others. By that time the impossibility of making the existence of the water-works compatible with the safe navigation of the river, and indeed with the safety of the old bridge itself, had become so apparent that none of the projects submitted contemplated the continued existence of those works, the removal of which seems to have been considered but forty years before as a thing to be wished, but not to be hoped. A scheme was entertained, however, by all the projectors, and fostered by the Committee, who eventually reported to the House in its favour, for making the river navigable by large sailing craft as far up as Blackfriars' Bridge by rebuilding London Bridge with head-way underneath sufficient to allow vessels of the size that depth of water could be found to float, to pass under the bridge with their topmasts standing. With this was connected a project for embanking the river to narrow and deepen its channel, and to form wharfs and warehouses upon the embankments; but the necessity which the great proposed height of the bridge (65 feet under the central arch above high-water level) occasioned, of carrying the approaches far inland at both ends, and the opposition of those interested in the water-side property to the embankments, had the effect

of preventing the measures proposed by the Committee for adoption, from being carried into effect; and nearly a quarter of a century more had elapsed before the subject was resumed with a successful result. By this time, indeed, circumstances had become materially altered; the matured steam-engine now furnished an economical and certain means of raising the water hitherto raised by the action of the current from the river through the medium of London Bridge water-works, rendering these no longer, as they had been believed to be, essential;—the establishment of docks had cleared the river of the larger classes of merchant ships that before encumbered its stream, and had thus made a greater extent of its surface within the town fit to be navigated by ships no longer desirable. The docks had also furnished corresponding facilities for landing and warehousing goods so as to render the projected embankments no longer so important to the uses of trade as they had been deemed when the banks of the river furnished the only quays upon which goods could be landed, and from whence they could be loaded for shipping. London Bridge was therefore rebuilt with but little more head-way than the other bridges above it, within the extent of London, afforded.

It may be considered fortunate that the obstacles alluded to, or such other difficulties as really operated to prevent the execution of the project entertained by the Parliamentary Committee in 1800, did intervene, since the inclination of the road-way upon the bridge, and of its approaches, would have been made so steep as to occasion almost as great an obstruction to the heavy

traffic of London to and over the new bridge, as the old bridge had formed to the navigation under it; and as, in all probability, Messrs. Telford and Douglass's project of a gigantic iron-framed bridge in one span of 600 feet would have been executed, had the scheme been then carried out, the defect on the bridge itself, if not in the approaches, would have been irremediable, except by the reconstruction of the bridge.

As the head-way given by Temple Bar has been the London standard for head-way under bridges over streets and roads, so was the rise or rate of acclivity, or,—to use a bad but now well understood term,—the gradient, of Holborn Hill the datum for the rise to and upon bridges; and such, or an average of one in sixteen, was the acclivity proposed by Messrs. Telford and Douglass to be given to the road-way to and upon London Bridge, according to the designs and suggestions laid by them, in the first place, before the Committee.[11] Mr. Mylne, the architect of Blackfriars' Bridge, in the maturity of his years, described "an outline of a plan," which he pro-

[11] The "observations" sent with the first designs stated the extreme descent on the bridge to be 2¼ inches in a yard, or 1 in 16 with a head-way under the centre arch of 65 feet,—the dimension shown on the engraved elevation;—those attending the later design, showing one opening of 600 feet span, state that the matters not excepted,—of which this is one,—"are supposed to remain precisely as in the former statements," though one of the parties who sent answers to the inquiries subsequently proposed by the Committee, understood Messrs. Telford and Douglass to intend the declivity of the carriage-way to be $\frac{1}{18}$th, which he (Colonel Twiss of Woolwich) considered unnecessarily flat! Mr. Telford lived to specify the rise and the consequent declivity of the road-way upon a bridge in a commercial city (Glasgow) of far less importance than London, to be 1 in 44.

posed to the same Committee, for a new bridge in the place of London Bridge, with a road-way "to the foot of the new bridge, and to the upper part of the centre arch thereof," .... "having an acclivity or rise *proper for carriages;*" by which expression Mr. Mylne says, in a note, "is meant 10 inches rise to a horizon of 14 feet long," explaining, in the same note, that "on Holborn Hill, it is, opposite to Shoe Lane, at the rate of 10½ inches, and opposite to Ely Place, 10 inches, or from 1 in 16 to 1 in 16·8!"

It is not indeed until within the last few years that the propriety has been recognised, even in England, of restricting the rise, and consequently the fall of road-ways upon bridges, and of the approaches to them, to the inclination that the sort of road intended would give at its angle of friction, or the inclination producing repose upon the materials used, and at which inclination waggons and heavy carriages may be safely allowed to go down without skidding, or dragging a wheel. This may be taken at about one in thirty-five upon broken stone or Macadamized roads, and at one in forty-five or fifty upon ordinary pitched paving. The road-ways of Westminster and Blackfriars' Bridges, with inclinations of one in fifteen, have consequently been recognised as nuisances, and the road-way to the former has been already reduced, by a most skilful operation, to one in twenty-four, and the latter bridge is in course of preparation for a similar process, as the best approximation to what is desirable that can be obtained under the peculiar circumstances of these constructions.

It may perhaps be worthy of consideration, whether in

such cases as those last referred to the near sides of the road-ways might not be pitch-paved to ease the up-draught, and the off-sides laid with broken stone, to give more friction in the descent, with further advantage.

Reverting to the projects submitted to the Select Committee of the House of Commons in 1800, for removing Old London Bridge and providing a substitute for it which should give access to the part of the river above the site of the bridge to masted vessels of a certain class, and to the evidence and opinions given upon them, one or two observations occur which carry instruction with them. It appeared that, in connexion with his project for rebuilding the City after the great fire, Sir Christopher Wren had proposed to alter and improve London Bridge by obliterating the two northernmost arches, removing every other pier from thence, and reconstructing the upper works of the bridge with nine, or half the number of arches, upon the remaining piers. At that time not more than the two northernmost archways had been occupied by water-wheels, and the buildings on the neighbouring shore having been destroyed by the fire, the approach on that side might have been raised to the extent that the design required without any great difficulty; whilst the property immediately adjoining the bridge on the south side was of but small comparative importance for some distance inland, and the approach on that side might therefore have been raised without much inconvenience; but Sir Christopher Wren was in advance of his age, and his design for improving London Bridge was thrown aside

with that which he proposed for rebuilding the City; the water-works continued to encroach with their wheels upon the water-way, until five archways of the bridge at one end, and two at the other, were occupied by them, and until the action of the water, setting with a fall through the impeded and contracted passages, had rendered the old structure as insecure in itself, as it had always been dangerous to the navigation of the river. The second Report of the Select Committee[12] says that Parliament had granted £82,000 between 1758 and 1765 for the alteration and improvement of the bridge, and that, nevertheless, the average annual expense of repairs for the last ten years had exceeded £4,200. According to Mr. Mylne £90,000 had been laid out, and the revenues of a large estate expended, to preserve the bridge; whilst Mr. Ralph Dodd,—who is reputed the original projector of Waterloo Bridge,—says, "the wretched fabric is held together at a great expense (above £4,000 per annum), which, if left to itself for two winters, would inevitably tumble into ruins:" and recurring to Mr. Mylne's statement, "its figure and foundations contain the seeds of sudden, not gradual, dissolution."

It cannot be more strongly inculcated than in the lesson here taught, that such evils as those which were connected with Old London Bridge should be traced to their source as soon as they are perceived, and be at once removed with their causes, since nursing them and

[12] Ordered by the House of Commons to be printed, 11th July, 1799.

tampering with them only tend to perpetuate the mischief without diminishing its cost.

The most magnificent looking designs submitted to the Committee were those of Mr. Dodd who is quoted above, one of them being an extension of the project of Sir Christopher Wren; but Mr. Dodd did not improve in every point in which his design differed from Sir Christopher's. The latter, as already intimated, extended the north abutment through the space occupied by the archways, where the water must have been always slack, and made nine arches over the water-way; but Mr. Dodd placed the northernmost arch of his design for a seven-arched bridge upon the site of those obliterated by Sir Christopher; thus throwing the whole construction over to the north side, and hiding one arch behind his embanked quays. The old piers were to be used, and the central arch was to be of iron, with an elliptical opening of 300 feet span, and high enough "to admit ships of 500 tons burden to pass under it." Mr. Dodd determined, however, upon subsequent inspection, that the old foundations were not to be trusted, and suggested a new iron bridge, of three such arches as that above referred to, if an iron bridge should be preferred, but he himself strongly recommended a stone one, and according to his second design, in which the abutments were brought forward, and the number of arches reduced to five, the central arch being of 160 feet span, and,—according to the written description,—80 feet high, though the illustrative print shows the arch as a segment less than a semi-circle. The road-way to this was to have been

of the monstrous rise of one in fifteen from St. Thomas's Street on the south, and from Eastcheap on the north, to the centre of the bridge, making a plane on the City side of the steepness of Holborn Hill, and nearly three times its length; whilst that on the Southwark side would have been of the same degree of steepness, and even yet longer!

Mr. Dodd's first design exhibits considerable ingenuity in providing resistance to the thrust of the great iron arch, which is effected by a series of low elliptical arches under the upper semicircular arches which carry the road-way from the abutments on to the great central arch, the navigation of the river for craft requiring lofty head-way being well provided for by the great archway. The iron arch, too, presents an extreme degree of massiveness, that would have accorded well with the masonry constructions in conjunction with it, though the coupled columns in face of the piers, and their frittered entablature surmounted by a frippery balustrade, detract much from the generally striking effect which the composition otherwise produces. The design for a bridge of masonry in five arches is a mere common-place series of ill-shaped arches overlaid with architectural puerilities, which Mr. Dodd intended to be indicative of "national grandeur," whilst both the designs, or rather all the three, by this author,—as one design of three iron arches is implied,—are liable to utter condemnation because of their inapplicability to the site through their loftiness, and the consequent impracticability of their road-ways; but this must be attributed rather to the general inten-

tion of the time than to any exaggeration on the part of this particular projector.

Mr. Mylne sent no drawing of a design for a bridge, but he gave a general description in writing, from which may be inferred a something not unlike Mr. Dodd's five-arched bridge of masonry, and with road-way and approaches "*proper for carriages,*" according to Mr. Mylne's estimate of what propriety in that respect was.[13]

Mr. Dance, the architect of the gaol of Newgate, and at the time Surveyor to the City of London, proposed to make the space between London and Blackfriars' Bridges available for shipping without raising the lower of the two bridges so high as to render the road-ways to and over it "an insufferable nuisance," (to use Mr. Dance's own words,) in an ingenious and not altogether ineligible mode, though the project was not received with the same degree of favour which the Committee deigned to those designs which embodied the favourite scheme of leaving the river navigable under the bridge for ships with topmasts standing. Instead of one lofty and almost inaccessible bridge, Mr. Dance proposed "to erect two low and level bridges, at the distance of 300 feet asunder, with a draw-bridge in each for the passage of ships, and by the alternate opening of these draw-bridges at suitable times of tide, under the regulation of a proper officer, and signals, by hoisting flags, or otherwise, might be adopted, the public would always have an uninterrupted passage over one of the bridges,

[13] See *ante*, p. 64.

whilst the other was open to admit the ships,"[14]—which were to be moored in the space between the two bridges to await the tide and their turn. In addition to the ingenuity displayed in the arrangement of the design generally, and the taste evinced in the composition of the twin bridges with one another, and with the proposed approaches, the design of the bridges themselves is favourably distinguished from the other designs for the bridge by a dignified simplicity far more consistent with true grandeur than Mr. Dodd's "columns, niches, trophies, statues, and other ornaments." In aiming at economy in the structure and in the approaches, however, and to keep the bridges themselves level, Mr. Dance's bridges were so much depressed that the springings of their elliptical arches are below high-water level, and the general head-way for ordinary river craft was altogether too low, and it may be questioned whether the piers to the draw-bridges were of sufficient substance to resist the thrust of arches so flat as those proposed. Moreover, the semi-cylindrical attachments to the faces of the ordinary piers injure the composition by the heterogeneousness of their forms, and by cutting off the springings of the arches, whilst the cylindrical towers of the draw-bridges are even more objectionable, as they stand out of the line of the faces, though indeed, by thus standing out, they allow the arches to spring from their true horizontal beds. The position assumed for the

[14] Page 79 of Appendix (D) to third Report from the Select Committee upon the Improvement of the Port of London. Ordered to be printed, 28th July, 1800.

sister bridges and the pool between them was selected in such manner as to bring the Monument on Fish Street Hill into the centre of the vista northward, and Mr. Dance, with great good taste, introduced a lofty and well-proportioned obelisk at the southern end of the pool to balance the column on Fish Street Hill, the north end of the pool being occupied by an equestrian statue. It may too be questioned whether the end aimed at would have been perfectly answered in the manner proposed at all, and certainly it could not have been even well effected with but one draw-bridge in each of the main bridges, as Mr. Dance appears to have himself suspected, whilst there must have been always considerable practical difficulty, and no inconsiderable expense, in turning the tide of commerce over from one bridge to the other in so thronged a commercial thoroughfare as London Bridge must be while London is what it is.

General Bentham, an eminent engineer officer, laid before the Committee a project of a single bridge widened in the middle to admit of a chamber capable of receiving a ship between two draw-bridges, which were to be worked in the manner proposed by Mr. Dance in his project of a pair of bridges with a pool or basin between them. According to General Bentham's plan, the working of the draw-bridges, and the consequent interruption of the line of thoroughfare, must have been incessant, if ships had been required to pass at all, and it may be further objected to it that such a restricted space as a mere lock-chamber in the stream of a rapid tide-way would have occasioned great difficulty in laying a vessel

in without severe concussions, from which both ship and bridge-works must suffer, though the plan had this advantage over Mr. Dance's, that a vessel was passed in one operation, and the trouble and delay of mooring in the pool avoided. As a design for a great public work, however, General Bentham's is unworthy of a moment's notice, its only interest arising from the project of passing ships through without interrupting, except by turning, the thoroughfare upon the bridge.

Mr. James Black proposed a granite bridge of three unequal arches of the Neuilly "complex" form. The design is remarkable alike for the boldness and real grandeur of the project, for the feebleness of the masonry of the proposed structure, and for the bad taste of the composition and arrangement in detail; and it may be further remarked that Mr. Black went even beyond his brother projectors in the rise he gave to the road-ways of the approaches, which was one-fourth steeper than Holborn Hill, or at an inclination of one in twelve, the rise upon the bridge itself over the first and third arches being at the rate of one in eighteen. The boldness of the project consisted in the proposal to build a granite bridge with arches of 220 and 240 feet span, rising but one-fourth their span respectively, as nothing approaching that degree of magnitude with similar proportions had been any where or at any time attempted. Whether it was deemed impracticable, or that the Committee were led away from the consideration of any other by the captivating effect of Messrs. Telford and Douglass's project for an iron-framed bridge of 600 feet span in one opening,

does not appear, for they simply remark upon Mr. Black's project, that Mr. Black proposed " a bridge of granite of three arches of an extent much exceeding those of any stone bridge which has been attempted in Europe," and proceed to call attention to the " obvious advantages which would be obtained if the communication could be effected by means of a single arch, as well as the magnificence of the proposed structure,"[15] overlooking altogether the " obvious advantages" of granite masonry over cast-iron framing. The great defects of composition and in detail in Mr. Black's design were remediable, whilst its real magnificence was inherent, and the design was consequently much more deserving of attention than the magnificent wonder which threw it into the shade. It is true that the Committee had already reported to the House in favour of an iron bridge, but the observations Mr. Black submitted with his design for a bridge of granite masonry, with reference to this Report, seem to deserve more attention than they obtained at the time. Having quoted the resolution of the Committee in favour of an iron arch, Mr. Black observes " that a frame-work of iron or a frame-work of wood will be subject to considerable vibrations; that vibrations are attendant on all structures, and are the primary cause of their decay and final ruin, wherefore structures which are by their form least subject to vibration are the most durable." " But it may be proper to observe also,"—Mr. Black continues, after citing an example that he considered a

[15] Supplemental Report from the Select Committee upon further Improvement of the Port of London, 3rd June, 1801.

proof of what he had said,—" that when the causes are complete, a frame-work may be subject by its form to instantaneous ruin,—that, on the other hand, solid structures of stone proceed by a gradual decay the common course of nature. Thus, although a frame-work may be endowed with its beauties of form, and be very compact in its construction, it cannot be considered as having the properties of solidity and durability of a body well proportioned and constructed of the best materials." [16]

Several designs for a bridge of cast-iron were proposed at the same time with the former designs of Messrs. Dodd, Mylne, and Dance. One was a design with three unequal openings or archways, by Mr. Wilson, who is described as the " architect of the celebrated bridge at Wearmouth, near Sunderland," but it was of far inferior merit in its composition to the latter work, and, indeed, it possessed but one merit, and that somewhat of a negative quality, that the ascent was of only 1½ inch in a yard, or at the rate of 1 in 24, though the height above the water was the standard of 65 feet, so that the approaches must have been extended longitudinally in proportion to the limitation of rise in the ascent. Three or four of the designs were by Messrs. Telford and Douglass, some being of three and others of five unequal openings; but all that are published are conceived and composed in the worst possible taste; the iron ribs in every case being segments of circles placed

[16] From Appendix to Supplemental Report last quoted.

between the piers without either impost or abuttal, and in appearance poised in the air upon acute angles between the sides of the piers. One of these copartnery[17] designs placed the greatest opening, and the highest part of the bridge, nearer to the north shore than to the south, to ease the ascent from the latter; thus taking the loftiest head-way away from the most navigable part of the channel for the very trifling economical advantage to be derived from a slight reduction of the extent of the southern approach. It was subsequent to the publication of these designs, and of all the rest before referred to, except Mr. Black's and General Bentham's, as well as of the Report of the Committee upon them, that Mr. Black's design for a granite bridge in three arches, and Messrs. Telford and Douglass's for an iron-framed bridge in one span of 600 feet, were delivered. The latter design produced so strong an impression upon the general mind that the Committee of the previous years re-assembled in 1801, and entered into an investigation of its merits, submitting a series of practical and scientific inquiries to those persons who were deemed most competent to give information and advice to remedy defects if such should

[17] It is somewhat remarkable that in the lately published "Life of Telford," written by himself, and edited and amplified by his friend and executor, Mr. Rickman, no mention is made of, and no allusion can be discovered to, Mr. Telford's connexion with Mr. Douglass, nor, indeed, to the designs themselves for London Bridge; and it is hardly less so, that in the written document submitted by Messrs. Telford and Douglass, with their designs, the former signs first, and designates himself "Surveyor," and Mr. Douglass, following, writes himself "Engineer," whilst they are designated jointly, by the Committee, "the Architects"!

be thought or be found to exist in the design. One of the persons applied to was General Bentham, who at once laid the inquiries, and the design to which they referred, aside, and applied himself to improve Mr. Dance's project in the composition already alluded to; and another was Mr. Southern, a contemporary practising civil engineer, who, disapproving of the principle of construction of the iron-work of Messrs. Telford and Douglass's design, suggested another mode, which he thought better adapted to answer the end in view. All, however, who expressed an opinion as to the practicability and advisableness of such a work as that proposed, (and almost all whose answers are given do so,) concur in the affirmative, how widely soever they may differ in the course of reasoning by which, or in the assumptions upon which, they arrive at the conclusion. Many of those who expressed an opinion in favour of the practicability and advisableness of the plan stated also their belief that an arch constructed according to it, was, in the words of the inquiry, " capable of being rendered a durable edifice." The diversity of opinion upon most of the matters of detail was, nevertheless, very great. Some treated the construction as that of an arch, and others considered it as a piece of framing to bear across; some preferred an elliptical form to the segment of a circle, which the design exhibited; some thought the widening on the plan from the centre towards the abutments a great advantage; others recommended that the bridge should be made generally wider, but not to extend the spur widening beyond one-third from each abutment;

and others again thought the spur widening not only unnecessary but really injurious, because of the great weight thereby brought into the haunches or spandrels without countervailing power in the middle, or towards and in the crown. It was a general opinion, indeed, that the haunches were overloaded with metal, whilst the devices for lightening them were various; and it was thought by some that the construction might have been greatly improved by occupying the height of the balustrade above the road-way with what should contribute to the strength of the work, even though it had the effect of placing a barrier between the foot-ways and the carriage-way, and even of dividing the carriage road-way into as many track-ways as the bridge contained ribs.

The plan of the bridge, according to the design of Messrs. Telford and Douglass, was that of the common class of road-bridge usually built in brick-work over canals, and of which there are yet too many existing examples on main lines of road, even about London. To save wing-walls, and to make a narrow bridge look wider than it really is, the faces are made concave, so that the parapets are convex towards the road-way; and in the design in question for London Bridge, the faces bent back from the abutments towards the centre in flat segments of a circle narrowing the road-way, or rather the breadth of the bridge upon the plan, from 90 feet at the abutments to 45 feet in the middle. The object in this case was to enable the bridge to withstand the concussions to which it was to be considered liable from the

masts, and even from the hulls, of ships intending to pass under, or being drifted by accident upon it. Mr. Jessop thought that the bridge might be made strong enough to resist any lateral bias without the spur widening, and at much less expense; and it seems clear that the bridge might have been made of the greater width of 55 or even 60 feet throughout at no greater expense than the widening from 45 to 90 feet would have involved, with a manifest advantage to the useful qualities of the work as a bridge in an immense thoroughfare. The form of the arch was that of a segment of a circle, whose radius was about 725 feet, the chord being 600, and the versed sine 65 feet, having haunches of enormous depth at the springings, as the inclination of the road-way, great as it was, did not take much out of the rise, after the vertical thickness of the work at the crown is deducted. An elliptical form, however slight the tendency to ellipse had been,[18] would have given a much more graceful appearance to the work than it could derive from a mere segment of a huge circle, whilst every shade of approximation to the ellipse would tend to lighten the spandrels, in which case the haunches might have been most beneficially made deeper,—not by

[18] The approximation to an elliptical form here intended has received an illustration in the iron bridge over the Lary, near Plymouth, by Mr. Rendel; a work in which the difficulties connected with that class of bridge composition have been met and mastered more effectually than in any other work of the kind with which the present writer is acquainted. A representation of this bridge will be found in the first volume of the "Transactions of the Institution of Civil Engineers."

placing the springings or imposts below the level of high-water, but by raising the road-way over them until the inclination should have been brought to what a heavy passenger carriage could be trotted over, and a laden waggon drawn over, without extra power on the rise, and without locking the wheels on the fall. Without an alteration to the effect last indicated, indeed, whatever other merits or attractions it might have possessed, the bridge would have been, in Mr. Dance's words, an insufferable nuisance. Moreover, the gradual wedge-like narrowing of the road-way towards the middle of a bridge a furlong in length, to one-half the breadth it possesses at the extremities, where alone there could be an escape laterally, would have led to inconvenience, and have produced disasters that do not appear to have been contemplated. The proposed inclination of the road-way is such that all classes of carriages must be brought to the same pace in ascending, keeping the lighter and swifter carriages in their ranks abreast of the heavier and slower; and as, in such a thoroughfare as that over London Bridge, the widest road-way that could be provided would be often, if not constantly, occupied across the whole breadth, the tendency of such an arrangement of the plan of the bridge is to wedge the ranks of carriages together as they advance towards the middle, and thus render the way totally impassable.

The foregoing remarks upon the transactions and propositions in connexion with Old London Bridge and the communications over and upon the Thames, having relation to both water-way under and road-way over it

within a great commercial city, bear either directly or incidentally upon most of the more important matters that require consideration in arranging for the improvement or reconstruction of a bridge, and—with the observations preceding them—for the construction of a new one. The mischief that may be done by a bad bridge is here shown in the clearest light, whilst the futility of attempts to alleviate evils without remedying the abuses which render them greater, is fully proved by example. The case illustrates also the necessity of looking beyond the present moment, and of weighing the probable, and even the merely possible, as well as the certain results of what is proceeding or confidently intended, with what is proposed and contemplated. Whilst the House of Commons, by their Committee, were still occupied in seeking for and considering designs and suggestions for rendering the Thames accessible for ships a few hundred yards higher up than it had been, to the permanent injury of by far the most important communication over the river,—for the sole benefit, as it now appears, of the owners of the water-side property between London and Blackfriars' Bridges, and of the immediate owners of lighters,—the docks were already in progress, whereby the river was rendered comparatively clear, property secured, and the wretched system of loading and unloading ships, bearing merchandise, by means of lighters practically abolished.

PRACTICAL TREATISE.

The practice of Bridges, or the application of the art of forming constructed erections as Bridges, involves certain important considerations preliminary to the execution of any work. These have reference to the situation, to the object to be attained, to the materials, and to the design,— this subject of consideration depending greatly upon that which next precedes it, and both being always and of necessity influenced by the last, but not least important subject,—the means at the disposal of the architect, whether local, mechanical, or economical; whilst all the subjects named are so intimately connected by their influences upon, as to require that they be considered in connexion with, one another.

In laying out a new line of road through a country, choice may often be made of the points or places for crossing rivers or other similar obstructions; and as the part of a road upon a bridge must be always much more costly than any other part of equal length, it may be worth while to go out of the direct line to places at which crossings may be best effected. It may be assumed

as a general rule that the best point upon a river at which to establish a bridge is where the stream is narrowest, and where the banks are raised enough to form natural approaches and are solid enough to form natural abutments to the bridge: but such rule is subject to many qualifying exceptions. Where a river is narrowest it is deepest and most rapid, and where the banks are high and consistent enough to form approaches and abutments to a bridge, the water will rise higher in floods than at any point lower down where it may have more room laterally. Unless, therefore, the crossing can be effected in one span, reach, or bay, so as to avoid a pier or piers in the water-way, it may be less expensive and more certain to build a longer bridge over a wider part, and to form artificial approaches and abutments, than to construct a sufficient and perfect bridge where nature has provided both approaches and abutments, and where the traject is shorter. It does not often occur, moreover, that both banks of a river are high in the same place; and, again, where one bank of a river is elevated or cliff-like, and of good consistence, it is very likely to have been the means of reflecting or throwing the water off to act upon the opposite bank, and so have itself become the head of a reach, or the concave face of a bend, where the stream is very generally widest.

When circumstances admit of a bridge over a river being in one span or reach, or of one bay, it is comparatively unimportant whether it be placed upon a bend of the river or not, if it be so contrived as to give sufficient head-way for craft where the channel is, or, in

other words, where the water is deepest, which in a bend is most frequently on the concave side. When piers in the water-way are determined to be necessary for an intended bridge, the site should be chosen where the course of the river is straight, so that the bridge may be placed at right angles to the thread of the stream, and that the piers may thereby intercept and divide the water in the least objectionable manner. In a running river the longest part of a straight reach may be left above the bridge with advantage, whilst in a tidal river the bridge should be placed in such manner as to give the stream at both ebb and flow all the advantage that can be obtained from the piers lying in the direction of the currents; and where, as it must be in most cases, the down stream is the strongest, and the ebb of longer duration, the proportion of the reach above and below a bridge should be determined accordingly.

Besides adapting the piers of a bridge to the stream, a straight part or reach of a river has this other important advantage,—that in such parts a river presents the most uniform section, and is less liable to be encumbered with shoals, or rendered otherwise inconvenient by deep pools, the presence of which always indicates unequal or irregular action in the current.

In choosing a situation for a bridge, reference must be had to the ground upon which it is to stand, both as to the abutments on the margins of the river, and as to the piers which have to stand in the water-way. Sound beds of primitive or of horizontally stratified rock, and well compacted chalk, are unexceptionable as substrata for

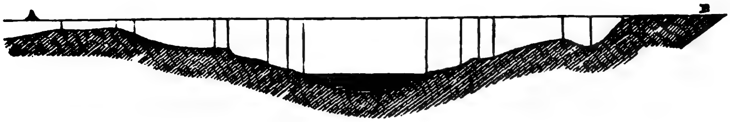
bridge constructions. Sound hard clayey gravel, through which water will not rise if tried with a head equal to what it may be subjected to in floods, forms an excellent base, and so do hard and not easily soluble or stiff clays, when in beds of considerable depth or thickness; but soft soluble clays, loose sandy gravel, and sand in any uncombined state, and subject to the action of water, are not to be trusted. It must, too, be always borne in mind that the contraction of the water-way which a bridge is almost certain to effect in a greater or less degree, tends to induce a more severe action upon the bed of the river than its substance can have been anteriorly subjected to.

Of course facility of approach to a bridge must not be overlooked in selecting a site upon which to build it; and it must be held essential to the goodness of a site that the approaches can be made direct and easy, the economical consideration determining whether it may be better to take up a more difficult position for the bridge, to obtain less expensive approaches, or to take up the best site for the bridge, and bestow whatever labour may be requisite to make suitable approaches to it.

Very generally, however, in towns and cities, and for the most part throughout old settled countries and in commercial districts, bridges must be built where bridges are wanted, and a bridge must be adapted to its site, be the site where and what it may.

The situation of the bridge being determined by choice upon exploration, or of necessity as last intimated, a careful survey should be made of it and of the proposed

line or lines of approach, and of the river over which the bridge is to be built, for the whole length, at the least, of the reach upon which it is to be placed, and the results of the survey laid down as a map or plan presenting an ichnographic outline or representation upon a horizontal section of lines raised vertically from every point of the surface of the ground; for it must be



obvious, upon an inspection of the diagram, that a plan of the site from A to B, laid down from measurements made along the bending line of the surface, would give a much greater length than a plan upon a horizontal section of lines raised vertically from the surface of the ground to the straight and horizontal line A B. The plan required is such as this latter description indicates. The irregularities of the ground must also be represented in diagrams such as the above, so that in arranging, designing, and estimating the intended work, the architect may have the means before him of ascertaining the exact dimensions yielded by the site in every direction. Such a diagram as the above is termed a section, and it should show, in addition to the irregularities of the ground, the various substances of which the ground is composed, and the thickness vertically of each substance as far as the object in view renders it necessary to ascertain it,—that is, until a proper substance upon which to work has presented itself. This may be ob-

tained by boring with a species of auger made for the purpose, the stem of which can be pieced out to almost any length, and which brings up specimens of the substances through which it is bored. If piers are to be built in the water-way, such an examination, or exploration rather, of the bed of the river, cannot be made with too great care and minuteness; and as facts are ascertained, they should be carefully and instantly minuted and laid down upon the section. Besides the transverse section of the river, which is the direction to give the longitudinal elevation of the proposed bridge and its approaches, a longitudinal section of the river, transversely of the bridge, should be taken to show the declivity of its bed, and to exhibit any unsound strata that may lie near the proposed site of the intended work.[1] Upon both sections of the river, the ordinary depth of water in it, the highest level it ever attains, and the lowest to which it falls, should be ascertained and recorded. It may be difficult at times to determine the height to which occasional or unusual floods have raised the waters of a river, but it is most important to ascertain the whole truth in that respect, and for the purpose of arriving at it the evidence of facts should be sought as well as that of persons and of records. The river should

[1] A long pointed iron rod may be used with advantage in the general examination of the ground, and of the bed of a river. The comparative ease or difficulty with which any rod may be forced down into a sound bed of clay that may be at hand, and into the bed of a river, or into the ground on its banks, will enable an observer to judge with tolerable accuracy of the soundness or unsoundness of the latter before the process of boring is undertaken.

be traced upwards; and in old countries, ruins of earlier works or existing structures on or near the banks will be often found to retain evidence of inundations by stains, or by the adhesion or deposit of weeds and other drift, in such manner as to be easily recognised by the observant. In new countries, trees and underwood, or brush, near flooding rivers, retain certain indications of the heights to which floods rise by the drift which hangs on their branches, and by the discoloration which intercepted scum and earthy matter that may have been held in suspension in the flood-waters occasion.

The object sought in the erection of a bridge is an uninterrupted road or way over or across a river or other obstruction in the most convenient manner and place for the traffic or uses of the road or way, and this must be kept steadily in view, though in seeking to attain it no other interest should be sacrificed, and nothing done to bar future improvement, nor should the removal of one obstruction be made productive of another. A river may not be liable to high floods, and it may be a matter of calculation that the piers or other substructions of a bridge built in a certain manner and of a certain extent will not hold the water up sufficiently to endanger them, but the water may nevertheless be dammed up in such manner as to interfere with the proper drainage of the land, even if low lands are not directly flooded by the waters of the river breaking in upon them. Again, a river may not be itself navigable, but it may be susceptible of being made so, or of being connected with an artificial navigable water-way; and care should be taken,

therefore, in aiming at the primary object of a bridge, not to erect what may be a bar to the establishment of an efficient way for commerce in another direction.

The approaches of an ordinary road-bridge, and the road-way upon it, may be adapted to the roads which it connects; for although it is desirable that the rise and fall upon a bridge within a town should be as easy as possible,—and ease in that respect may properly be sought, even at considerably enhanced expense in such a case,—it is not worth while to incur great expense to make the approaches to and the road-way upon a road-bridge easier in its inclination than that which the face of the country compels the adjacent and connected roads to assume, since carriages must be supplied with power of draught sufficient for the steepest inclinations of the roads, and the bridge may, without economical impropriety, present the average inclination of the roads which must be worked over to and from it.

The materials and the design [2] of a bridge are, of necessity, most intimately connected; and both, as before intimated, must be greatly influenced by the means at the disposal of the architect, and by the duty to be performed.

Bridges whose main constituent is iron are built to a great extent in England, because in this country iron

[2] By design or a design is intended, not the mere elevation of the faces of a bridge, but the arrangement and distribution of the whole work in its constructive details, and in its adaptation to the office to be executed.

is comparatively cheap, and the mechanical means required for its fashioning and fixing are easy of attainment; whilst the aim in most new countries must be, as it is indeed in many old ones, to make timber answer the same purpose, and to connect it in bridge constructions with the smallest possible quantity of iron.

All bridges, however, whatever may be their main constituent, and in what manner soever they may be formed or constructed, should bear upon masonry substructions, if permanence be desired; and in the absence of stone, brick-work at least should be used for piers and abutments. Piles of timber must be consented to, nevertheless, when better materials and the means of procuring and employing them are wanting. In very many cases a bridge must be utterly denied, unless both piles and abutments of timber be admitted, and it will then be the duty of the architect to render the material at his disposal as rigid as possible by framing, bracing, and otherwise combining and fixing, and to protect it to the greatest practicable degree from causes of decay. This may be said indeed in favour of piles in a water-way, that they give the resistance required in bearing piles or piers with less substance than piers of masonry or brick-work involve, and consequently act less injuriously upon the current.

A bridge of timber, of which the supports are piles driven into the banks and into the bed of a river, at short intervals, their heads tenoned into beams laid transversely, or connected by half timbers in like manner, secured to them as waling-pieces, with whole

timbers laid longitudinally upon the heads of the piles as brestsummers, and these covered with planks stout enough to bear across and to support the weight of the traffic, whatever it may be, is a simple and comparatively inartificial work, and one that admits of no great variety in design; sufficient strength in the timbers, and security in connecting the various pieces together, being all that is obviously necessary: but there may be a very great difference in the cost and power of endurance of such a construction, according to the mode of combining and connecting the work. Without skill, a bridge of the kind alluded to would consume a large proportionate quantity of iron in straps and bolts, whereas such a work might be put together with greater efficiency almost without iron-work at all. The same remark may be applied to all constructions of timber, but to none perhaps with greater truth and effect than to timber bridges.

From simply bridging with timbers from pile to pile across a water-way by transverse strain upon the timber, through the varieties of arrangement and combination of which timber is susceptible until it is hardly exposed to transverse strain at all, the extent to which design may and must be varied is very great. The beam or brestsummer bearing from pile to pile may be strengthened by means of corbelling pieces; the same results may be retained with a lengthened bearing by means of trussing, which may be done in many and various modes, none of which, however, exert, or should exert, any thrust upon the bearing points, and consequently simple brestsummer

bearings, corbelled brestsummers, and trussed brest-summers, require nothing to abut against, or as abutments;—all such combinations require merely props or supports as piles or piers. The American latticed bridge is in effect a system of longitudinal bearing beams or brestsummers, the beams having the depth necessary to long bearings given them in such manner as to relieve the pieces of which they are composed of transverse strain, whilst the bearing of the artificial beam is vertical and without extraneous thrust. With simple bearings, however, as in the preceding combinations, as the strength is increased, or the bay extended, the head-way must be occupied, or the trussed, or latticed beam must rise above the floor and limit the width of the road-way or divide it, or render artificial strength necessary to the transverse beams or girders. Struts and straining beams under longitudinal bearing beams or brestsummers to strengthen them, are but a rude approach to an arch with timber, and like an arch they exert a lateral thrust, and require abutments or points of sufficient resistance where the system of which they form a part terminates, as at the two ends of a bridge. Ramified combinations of the strut and straining beam by bolting up or trenail pinning, and otherwise connecting short pieces of timber in thicknesses, produce ribs possessing many of the qualities of an arch, and susceptible of being treated as arches according to their flexure and form. With these, abutments are of course essential, though in truth most combinations of timber for the bearing works of bridges tend in a greater

or less degree to the effect of a deep beam variously strutted and trussed, how much soever the effect of an arched construction may be incorporated. Messrs. Green's bridges in the line of the Newcastle, North Shields, and Tynemouth Railway (see Plates 11, 12, 13, 14, and 15) rely more upon the effect of the arch in bearing across than any other timber bridge extant; but even these bearings have much in them of the deep beam, which seems to involve the true basis for timber bridge constructions, although they may embody that of the arch also.

When arched ribs of timber, whether bolted up or trenail-pinned in thicknesses, or shaped and put together with abutting joints, are placed under a road-way, the load of the road-way and of the traffic upon it must be distributed as equably as possible by means of struts and braces, and through the intervention of a continued beam as a curb or discharging beam over every rib; and when ribs,—which then must be of the former class only, —are raised above the line of the road-way in the manner referred to for deep trusses or latticed bearing beams,— exacting also strong transverse beams, which, with the road-way, are to be suspended in the usual mode by means of iron rods,—the line of the road-way should, through a curb or by other ties continued longitudinally, be made to tie or string the bow which a rib forms, and be strutted down from the rib as well as hung up to it.

The celebrated timber bridge over the Rhine at Schaffhausen combined in itself almost all the varieties of which timber is susceptible to give strength and rigidity in long bearings; and such was the effect with which the

scarfing, trussing, strutting, bracing, bolting up, and suspending were applied in that work, that, according to the opinion of Mr. Telford, expressed in the Edinburgh Encyclopædia, if this bridge had been formed in a straight line between the abutments (364 feet), he could see no reason why the construction should not have carried all that need have been required of it without any intervening support.

But it is not only skill to give strength that the design of a timber bridge requires,—it demands also protection from the weather; and this demand, when it is responded to, seems quite to destroy all pretence to elegance and agreeableness of effect. The roofed bridge at Schaffhausen must have been a heavy and unsightly object, but it was really not inelegant in comparison with the American enclosed and covered timber bridges,[3] which are, moreover, rendered absolutely monstrous by the puerile attempts generally made to imitate upon them the appearances of constructions of masonry under circumstances which render such appearances both ugly and absurd. The bridge over the Delaware at Trenton seems to present a favourable exception to the ordinary practice in the United States in this respect; but even in that work the roof coverings to the curved ribs are run up to a point to deform it, without any apparent necessity. The design in this case might be greatly improved, and rendered indeed in a great degree unob-

[3] The boxed-up American bridges convey an idea of coffin cases for sea serpents.

jectionable, by adapting the roof covering to the form of the backs of the ribs,—and zinc or other cheap metal plates would give a ready and economical means of doing this,—and by raising the ridge of the roof over the spandrels to the highest level the ribs attain. When the bearing parts of the construction are entirely below the floor of the road-way, as in the case of the bridges before alluded to upon the Newcastle, North Shields, and Tynemouth Railway, the floor itself may be made to give, in a very great degree, protection from the weather to the main works. When the floor is longitudinally level it may be slightly draughted to the sides transversely, and being caulked with oakum and payed with adhesive bituminous substances (easily and cheaply renewable processes), and catch-water gutters and drains, with trunks from them, being formed and placed to lead the water down, roofing and casing may be dispensed with.

The decay of timber in bridge piles,—that is, in the piles which run through and act as piers do in masonry,—is most rapid between wind and water, or in that portion of the length of the piles that is neither always in water nor always in air, but alternately wet and dry, whether by the rise and fall of the tide, by the difference in height of summer and winter water where there is no tide, or by the splashing of the water by the action of the wind or by the run of currents. Where permanence in the work is aimed at, it is, therefore, desirable to make provision for withdrawing piles as they become insecure, and of substituting others in their places; though in the first place every thing should be

done that can be done to protect the timber from the injurious influence of the action alluded to, and so prevent decay as far as possible, or the trouble and expense of constant repairs may soon deprive the piles of their almost only advantage over piers of masonry. It has been proposed to drive piles, and to cut them off when driven at some distance below the lowest water level, and forming tenons on their heads to frame upon them large transverse timbers which may act as sub-transoms or sills, into which the bearing piles of the bridge may be stepped, and to which they may be secured until decay shall have rendered them unfit for duty, when the attachments may be withdrawn and the piles liberated to make way for sound ones. This must be at best but an insecure mode of founding a bridge, and would require to be aided by outworks, as cut-waters of driven piles, to and from which the bridge might be braced and strutted.

The injurious consequences to timber from the alternation of wet and dry render it imperative that in the design of a timber bridge the heels and springings of any ribs that may be used as arches should be kept above the level at which the tides, floods, or other occasion may raise the water over which it is to be placed; and, independently of decay, it is most important that the bearing constructions or upper works of a timber bridge should be above the possibility of being reached by the water, because, how mischievous soever flood-waters may become to a bridge of masonry or brick-work, such an one has a certain power of resistance upon

immersion, from the weight and mass of the substances of which they are composed; but timber being not only light, but for the most part specifically lighter than water, the upper works of a wooden bridge may be floated if not swept away, when a bridge of brick or stone might remain comparatively uninjured.

The support and bearing, both as to strength and durability, being provided for, power of resistance to the force of the currents must obtain attention. Piles should be driven in advance of the bearing piles to break the force of the stream or to turn masses of ice, and the piles should be all braced by diagonal, and connected by waling, pieces, which latter may serve as fenders also to protect the piles. The ribs or beams which carry the road-way must be well and thoroughly braced to withstand with more effect the action of the wind, and to check vibration whether from wind or from the passage of carriages. Where the bridge is narrow, and the span or bay wide, bracing within the width of the bridge may not be enough; and as cut-water piles may be generally carried out as far as can be desired without materially increasing the injurious effect of an obstruction upon the current of water, and the transverse beams may be run out in like manner without inconvenience, efficient means of strutting and bracing against the action of the wind may be thus obtained.

The principles which govern a bridge of carpentry are those upon which an erected bridge of iron must be composed. Iron bearing piles seem to have been considered nearly if not quite out of the question; and as

that metal when cast is brittle, and cast-iron piles are liable, therefore, to be broken in driving, and to be fractured by severe blows afterwards if they escape in driving, and cast being the only condition in which iron could be economically used at all, it may be thought hardly worth entering into any disquisition upon its use in piles. There may be circumstances and occasions, nevertheless, demanding a system of bearing piles which shall possess greater powers of resistance to external actions, and be more durable than timber alone can be rendered. Iron piles may be cast in lengths, and hollow to receive a whole length pile of timber as a core, the metal casing having spigot and faucet ends to make joint for the exclusion of the water, and having also all necessary cups, bosses, and flanges, for stepping, abutting, and bolting up the braces and struts, and to which to secure waling-pieces and fenders, which last should, perhaps, be of timber in any case for the sake of craft navigating the waters, and to break a blow and modify its action upon the more rigid pile by the softer and more elastic texture of the fender.

In many shallow mountain streams, which become torrents, and by swelling render it necessary to extend a bridge so as to carry it beyond a single bearing or bay, and where piers of masonry or brick-work would be insecure in themselves, or operate otherwise injuriously from the mass they must offer in their own bulk, such a system of timber-cored iron piles as that above suggested might prove advantageous. Indeed, the bed being in most such cases rock, or rocky, and so that piles could

not be driven with effect, the piles might be stepped into the rock upon dead or squared ends; or sleepers might be let down, where the stratum is broken, to a well compacted bed, and the piles stepped into them. Where piles of timber are liable to be attacked and rapidly destroyed by insects, it may also be worth while to adopt some such system of cored iron piling, though it must not be overlooked that there are influences to which the metal itself will succumb much more rapidly than the naked timber.

If circumstances render it absolutely necessary that the abutments of a bridge be formed of piling, such abutments should be coffered, and the coffers waled, braced and tied across most securely, and packed with concrete, or with chalk rubble rammed down to such a degree of compactness that the water may not penetrate the mass. The work should be made in this manner, of substance and strength sufficient to resist and hold up against whatever thrust it may be liable to receive from the bridge, or pressure from the embanked approaches, to the effect that the abutments shall hold the bridge independent of the weight of earth against it on one side, and that the embankment, in like manner, shall not require support from the bridge through the abutments. This is equally applicable, however, to bridge abutments of all kinds and under all circumstances;—they should be always of sufficient strength to hold the bridge and the embanked approaches to it entirely independent of one another.

Cast-iron bearing bridge constructions, again, must be

considered as partaking more of the beam than of the arch, though much valuable assistance may be derived with iron, as with timber, from the power of applying this material in the arched form, and indeed many important works have been executed in which the properties of the arch have been calculated upon almost exclusively. Nevertheless it will be found upon analysis of the design of almost all iron bridges, that the bearing is secured by the approximation made to the beam, although the resistance to compression has been carried through an arched form. The metal plates forming the parts of the presumed arch are, and must be, bolted up and so connected as to make a beam more or less cambered, and this is stronger or weaker according to the greater or less effect with which the spandrels are filled in and secured to the ribs and curb plates, and the trussing of the beam provided for through the latter: indeed, the best and most consistent iron bridges,—and the same may be said of timber bridges,—are those in which the work is so combined that the bearing might be trusted even without lateral abutments. The rib suspension bridge, whether the ribs be of timber or iron, is always supplied with the means of giving a tie, in the manner of a string to a bow, in the construction of the suspended floor; and the propriety, if not absolute necessity, of this should not be overlooked in the design for a work upon that plan.

Mr. Telford has conceded the credit of introducing iron as the main constituent of a bridge to Mr. Thomas Farnolls Pritchard, of Shrewsbury, who appears, from

Mr. Telford's Report upon an inspection of original documents, and from Memoirs published in the Philosophical Magazine,[4] to have proposed the use of iron in bridge constructions in the year 1775. Mr. Pritchard's earliest proposition upon the subject was to set up cast-iron ribs to act as a fixed or permanent centre to a back arch of masonry. The drawing of this bears date 1774, and the next, which was for a bridge of cast-iron between Madeley and Broseley in Shropshire, is dated, as above, 1775. The design for this latter seems to exhibit the germ if not the substance of all that has been done since. It exhibits a deep beam hollowed out to the general form of an arch-way and pierced in the spandrels, but in such a manner as to preserve a perfect connexion between the back of the beam and its lower face or soffit. One peculiarity in the design of the iron-work shows clearly, however, that the author's idea was to give the effect of an arch, by the combination he has introduced of arcs of varying radius, as in the inner and outer arches of Perronet's Neuilly bridge, which was at the time in course of execution. Mr. Pritchard's next published design is that of the bridge over Coalbrook Dale, the first bridge erected with cast-iron bearing or bridging constructions on the principle of compression, and the first work approaching the same degree of magnitude,—the opening spanned being 100 feet,—that had ever been executed in metal at all. This work illustrates the impropriety of imposing upon comparatively thin metal plates or ribs, in arches,

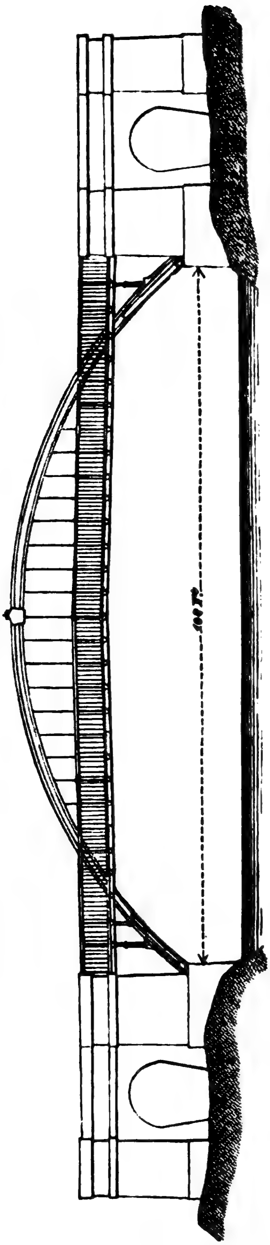
[4] Vol. xi. p. 183.

the various pressures to which arched constructions of unyielding substances placed in the mass, with their lateral faces and beds in close contact throughout the whole extent of the bearing, are safely enough submitted. It is true that the defect which showed itself was altogether independent of the iron bearing constructions, since it arose from the abutments being allowed to press upon them; but the abutments could not have been allowed to press upon the ribs, had it not been supposed that the arched form conferred upon them a power of resistance which the metal plates were not capable of exercising.

With reference to this example, though not, it is contended, for the true causes of the objection he states, Mr. Telford composed his first iron bridge with a combination of the arch-formed rib and of the cambered beam, introducing, as he truly remarks, more of the principle of timber trussing than of masonry; and this principle should be at least incorporated in forming the design of the upper works of a cast-iron bearing bridge,[5] though the arched form of construction be that to which the work may have the greatest apparent tendency.

The bowed or arch-formed iron rib applied to the suspension of a road-way which is made to tie or string

[5] The credit of carrying out this idea of trussing bearing beams by means of raised ribs, and of applying the principle of suspension in connexion with it in metal, seems to be due to Mr. Leather, of Leeds; but the rib suspension bridge had already been executed in timber in the well known example, already referred to, over the Delaware at Trenton in the United States of America, before any of Mr. Leather's iron arch or rib suspension bridges were constructed.



SKETCH OF MONK BRIDGE OVER THE AIRE AT LEEDS.

An iron arch or rib suspension bridge.

the bow in such a manner as to relieve the abutments of their thrust in some degree, is one of the least objectionable modes in which iron can be applied in construction, with reference to its liability to expand and contract; and the manner of construction here indicated involves one of the least expensive modes in which the metal can be used for wide spans and low head-way underneath, whilst it gives a more firm and rigid road-way than any mode of suspension from chains has yet been made to yield; though it is true that no just comparison can be instituted between the extent that has been spanned by the one and by the other mode. Notwithstanding the tie that may be obtained from the road-way to the raised and fixed rib suspending it, such a construction must not be treated as a mere beam, but it must have

abutments, from which to spring the ribs, of sufficient power of resistance to compel the expansion of the metal to act upwards and so to raise the road-way as much as the expansion of the ribs may exact; and this will be found to give the means of compensating also the expansion of the metal upon the line of horizontal tie, under or upon the road-way,—that is, if the metal in the one and in the other is well balanced with reference to that effect. The planking for the floor of a suspension bridge being of timber for the most part, it is adapted better to submit to vibratory or oscillatory action than if the basis of the floor were of metal; but with the fixed rib any provision to this effect ought to be unnecessary except for economy. A combination of timber and iron upon this principle might, however, be carried to some extent with considerable advantage in an economical point of view. Bolted up or trenail-pinned ribs of timber might be backed or capped with iron plates having rabbeted heading joints, and slotted holes for the suspension rods made through bosses which should be capped by the heads of the rods, and cast metal springing or heel plates would obviate the difficulty generally found in giving an efficient step for the heels of timber ribs, and so as to carry their thrust down to, and well to distribute it across the face of masonry abutments. Care must be taken at the same time to prevent water from running down the ribs into the cups in which the heels are thus stepped, or the result would be the rapid decomposition of the timber in those parts, and the destruction of the bridge, how well soever

the work may be capped, caulked, payed, or otherwise protected in all other places.

The flexible suspension bridge, as contradistinguished from that class of suspension bridge in which the road-way depends from and hangs upon a rigid line or rigid lines of wood or iron, as in works of the kind last referred to, requires a different mode of treatment in the design of all the primary parts as well as in the bearing over or across of the road-way. Flexible suspension bridges, or road-ways suspended from flexible lines, will be adopted only as temporary expedients, or in cases where the space to be carried over is greater than can, economically speaking, be effected with rigid erections. A suspension bridge may be readily and cheaply established as a temporary expedient with considerable advantage in most new countries, and various substances may be employed to form the ropes or cords from which to suspend a platform for a road-way. Long and flexible twigs, possessing many of the qualities of the ratan, and the ratan itself, may be put together in chains like wrought-iron rods, but with lashed instead of welded eyes; many trees and shrubs afford a strong fibrous bark capable of being twisted up into a rope,—the *currajong* of the Australian woods is an example of this,—some fruits yield a strong elastic fibre, as the cocoa-nut, from which the well known Indian coir rope is made; and rope once made may be readily manufactured into cables of any degree of strength required, whether the ropes be composed of vegetable fibre or of animal tissue, as ox-hide. The strength that may be ne-

cessary for the loads to be passed over a required bridge can be easily ascertained, and the make-shift cables may be put to the proof by suspending three or four times the weight to be carried, including of course the road-way and its attachments, upon dry ground. Indeed it is desirable at all times, when the materials to be used are novel, that proof should be made in every way by fitting up and testing the works of an intended bridge under circumstances which may render default on their part productive of no serious mischief; and this is more particularly necessary in the case of materials, such as ropes and cables, which are acted upon by tension, and whose strength depends upon circumstances beyond any calculation except what may be made from actual experiments. Such experiments must also be easy, for wherever the means exist of forming a suspension bridge at all, the means must be forthcoming of propping the cables, whether it be standing trees, trestles, or piers raised of stone or brick for the purpose, whilst trees, rocks, or other natural substances upon or near to the surface of the ground, or, in their absence, stiff stakes as piles driven into the ground, will give the means of securing the ends of the cables to counterbalance the road-way and its load.

In designing a suspension bridge to be executed *extempore* with such materials as may be at hand or easily obtainable, experiment having determined the quantity, substance, or bulk of the main constituent requisite to carry over the required space, recourse should be had to experiment to determine also the points at which the

props should be placed with reference to the distance between them, and the positions of the moorings for the ends of the cables behind them, their true places being where they bisect equally the angle formed by the cables of suspension when loaded. As both the moorings and the props are all-important in such a matter, if the locality proposed for a bridge should furnish peculiar facilities for either, it may be worth while to take up positions for the others that would not be chosen but for such facilities. Except for the most temporary purposes, however, no moorings should be adopted which, from their position or otherwise, do not permit access to the attachments of the cables, and the means of keeping them dry and in air; and it is obvious that no position should be taken up for the props, in which they would not be, or could not be rendered, perfectly free from danger from floods or otherwise, whatever they may consist of, or in whatever manner they may be erected.

Iron is used in the form of wire, which is made into cables of the sizes and strength, and of the whole length required, and it is wrought into links, bars, plates, bolts, and couplings of various forms and arrangements, and these, or some of them, are put together to form chains of the length required, whilst the requisite strength is generally obtained by multiplying chains in the bearing.

The great desideratum in all structures,—rigidity,—or what workmen know by the term stiffness, has not yet been attained in chain suspension bridges,—that is, in the part of a bridge for which alone the other parts are made,—the road-way; and it is a moot point how this

quality is to be secured.[6] It must be aimed at, however, in every design, and its attainment may be considered of even greater importance than the protection of the materials from decay, for these can be repaired and restored as occasion may require; but in the absence of sufficient stiffness to secure it against the action of tempests, the road-way of a suspension bridge is exposed to almost sudden destruction, or to damage beyond the possibility of repair short of re-construction. The serious alterations in length, which variations of temperature occasion in long metal rods and chains, and which are constantly taking place, seem to render it quite impossible to make perfectly rigid a road-way that is suspended by rods from chains, but much may be done by horizontal and vertical bracing both above and below the road-way itself, and by guy rods and chains to keep it steadily horizontal; in which condition, or position rather, it must be least exposed to be acted upon with injurious effect by the wind. There is great reason to believe, however, that the road-way receives its first impulse to motion from the chains themselves, in cases of the kind alluded to; so that the first step towards diminishing the influence of the wind upon a chain bridge seems to

[6] Mr. Rendel has recently re-constructed the road-way of the Montrose chain suspension bridge almost wholly of timber, and has succeeded in giving to it a great degree of stiffness by a judicious combination of framing and bracing within the floor itself, and independently of the rods and chains by and from which it is suspended; but the advantage appears to be obtained in some degree at the expense of the head-way underneath, and this in many cases is of great importance.

be to diminish the extent of flat surfaces exposed by the chains to the action of the wind. The substance or section of metal required in any case may be made to expose less of surface to the wind in one bar, or linked series of bars in one length, than it can be compressed into in more than one bar or series of bars; so that the smaller the number of chains, with the same effect as to strength, the less liable is the bridge floor suspended from them to be influenced or acted upon by the wind. But it is not alone on account of the amount or extent of surface exposed to the wind by few chains, than by many chains having the same transverse section collectively, that the greater number have been deemed objectionable. Separate chains of metal of great bulk and considerable extent, as in a chain suspension bridge, are found not to be acted upon by heat to exactly the same extent within or at the same time; so that one of a bundle of chains, having a duty imposed upon them collectively, may, by the greater or less expansion or contraction of some of them, throw the duty to be performed upon a part of the whole number. If these are sufficient, the remainder are unnecessary, and if they are not so, they fail, and throw the burden wholly upon the more or less contracted or expanded chains, which for the same reason should fail also, and the whole work thus fall into ruin. Hence it would appear that a greater amount of strength is necessary, and consequently a greater quantity of metal, as well as a much greater extent of surface, in a plurality of chains to do the duty of one, with certainty of effect, than the duty to be performed

requires,—since, in truth, every one of the number so employed should be competent to perform it alone. It is true that a bar, link, or plate of smaller substance may be wrought more perfectly, and be made thereby to sustain a greater weight in proportion to its substance than a large one can, where the means of working are not very perfect; and indeed there is a limit, beyond which even the most efficient means in use of working iron will not go in giving its full value to the cohesive powers of the metal. What is true of wire,—the smallest form in which iron is used for the purposes contemplated,—seems, however, to be true of bars or rods of larger bulk, when called upon to perform the duty together, which might be safely imposed upon their aggregate in one consolidated mass. The condition of wire is the strongest as well as the smallest form in which metal can be placed, substance for substance; but it is not found that a long cable made of a thousand wires banded together, possesses a thousand times the strength of a single wire (every wire being of the same strength), whether it be from the practical difficulty of drawing them all alike straight, and then holding and straining them all alike, or from other causes. More than a thousand wires must be put into a cable to enable it to sustain a thousand times the weight that may be entrusted to a single wire; but if the means existed of making a cable of the same quantity of metal the thousand wires contain, wrought and compacted as the substance of one wire is, such cable would possess the full strength of a thousand wires, and expose infinitely

less surface to the action of the wind than the same substance in separate wires would expose. As far as the means of thoroughly welding and working the iron in bars, links, and couplings may be at hand, or can be procured, so far it appears should the consolidation of substance in the chains of a chain suspension bridge be carried, but no farther.

The supports of a suspension bridge must of course be adapted to the weight to be borne, and to the required endurance of the structure, but the duty required should be limited to resistance to compression alone, or the power of sustaining the weight, such weight being imposed upon it vertically. The equal bisection of the angle formed by the chains in the main bay with the back chains is intended to produce this effect, and it must be obvious that any divergence tends to produce a strain upon the prop in the direction of the greater side of the bisected angle.

The props or bearing uprights of a flexible suspension bridge may be of timber or of iron, or, as they generally are in works of any extent and pretence, of masonry, or of a combination of brick-work and masonry. Timber may be driven as piles, or framed down into sleepers bedded on brick-work or masonry substructions, and strutted and braced beams as transoms being framed down upon the heads of the piles,—with or without templates or struts as the case may require,—the cables are laid upon rollers set upon the transoms: cast-iron uprights with spreading bases may be in like manner stepped into stone curbs, plinths, or sleepers upon

solid substructions, to carry the cables or chains, and these must be well connected across, or transversely of the bridge, and braced, that they may effectually withstand any vibratory motion the chains of the suspended platform may receive: and constructions of masonry, or of brick-work and masonry combined, as bearing piers or towers of whatever form, must have substance enough to bear without yielding the weight to be imposed, and mass enough to withstand the vibratory action, or rather to be the fulcra upon which struts, braces, and guys may take their bearing or their hold, to prevent the generation of such action. Indeed it does not appear but that a system of vertical timber bracing between the sills and curbs of the road-way and the main chains, commencing by an abuttal on the podium of the piers or suspension towers, may be introduced in connexion with transverse bracing from main chain to main chain in the deep parts, or where the head-way would not be affected by it, with economy and advantage. Such a system as that here suggested would adapt itself readily and without derangement to the expansion and contraction of the chains and rods, and tend materially to stiffen the road-way, whilst it would check any approach to the undulations, which, without a system of vertical bracing, are found to follow the vibration of the road-way in tempests. Vertical bracing under the road-way occupies head-way below, or renders it necessary to raise the bridge so much higher to preserve it, without assisting in any degree to steady the chains and give

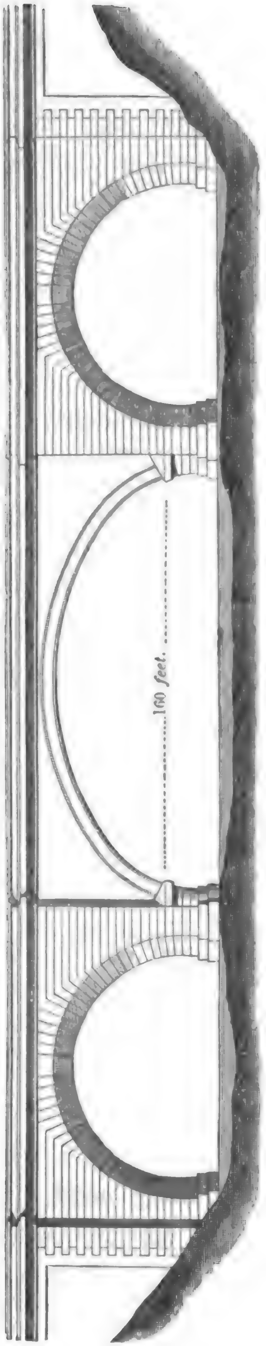
them the aid of the road-way in resisting impulses to motion.

For the reason last stated in objection to vertical bracing under the road-way of a chain suspension bridge, the plan proposed by Mr. Stevenson of Edinburgh (see Plates 104, 105, 106, and 107), of laying the chain under the road-way, is objectionable; but in cases where no inconvenience would result from this, it is a plan that has nothing to recommend it before the erected rib suspension system, but, on the contrary, the mode of construction it involves is less sound, the result is far less sightly, and certainly the plan presents no advantage, as far as economy is concerned, over that other and more eligible mode of construction which is here contrasted with it. Both plans or modes of construction are limited in the extent of span that can be properly embraced by them, and cannot therefore be applied in very many cases that are greatly within the powers of the chain; but there seems to be no reason why the erected rib should not be applied to spans exceeding 200 feet, whilst Mr. Stevenson considers about 200 feet to be the limit which the making up of the road-way of his bridge, and the enlarged angle of its suspension, set to the extent of span of which the plan is capable. The larger of Mr. Leather's rib or bow suspension bridges over the river Aire at Leeds exceeds 150 feet in span, and the ribs which carry the suspended aqueduct of the Aire and Calder Navigation, more recently erected by Mr. Leather at Stanley Ferry, also span 150 feet, whilst the timber ribs of the

central bay of the bridge over the Delaware at Trenton are of 200 feet span.

The erected rib suspension system may therefore be safely recommended to the extent of 200 feet span, and it may without danger of failure, if properly executed, be carried still farther; but for single bays of 300 feet and upwards recourse must perhaps be had to the chain, or to the flexible suspension principle. The Menai Bridge upon this system embraces 550 feet; the Clifton Bridge erecting over the Avon near Bristol will reach 700 feet; and the road-way of the wire bridge over the valley of the Sarine at Fribourg in Switzerland, by M. Chaley, a French engineer and contractor, is carried over a void of upwards of 800 feet, whilst the distance between the bearing points of the chains exceeds 870 feet (see Plates 98, 99, 100, and 100 *a*). To this it may be added that Mr. Telford did not hesitate to project and recommend a chain or rather jointed rod suspension bridge over the Mersey at Runcorn of 1000 feet in length between the bearing points.

In designing a bridge of masonry or of brick-work, involving piers in a water-way, some apparent contradictions have to be reconciled. The piers should have the largest possible base, and they should themselves occupy the smallest possible space consistently with their own efficiency; the level of the road upon the bridge should not be raised above the level of the roads which it connects more than is absolutely necessary; and the head-way under the bridge, both for the passage of the waters and for navigation, should be as high and



BRIDGE OVER THE HERAULT AT GIGNAC, NEAR MONTPELLIER.

Example of a bridge with large central arch, and smaller collateral bays.

clear as it can be made consistently with the important objects of the safety and utility of the road-way over it, whilst the arches will, in almost all cases, be required as flat as they can be built, and at the same time it will be desirable to render the thrust upon the abutments as slight as possible.

Bridges of the class, or built of the materials, now under consideration, consist of arches resting upon piers, and sustained or held together by abutments, or of a single arch resting upon and sustained by its abutments. The desirableness of adhering to the arrangement indicated by the latter form has been already pointed out, and the reasons for it stated; but when the existing and required water-way exceeds 200 or 250 feet, one arch can hardly be contemplated, though where there is height enough to allow of an arch of magnificent dimen-

sions, with a greater water-way than can, with prudential regard to economy, be wholly spanned by masonry, it will be better in every respect to span the main body of the stream with a great arch, and to occupy the remaining space with two arches smaller than, and of different form from, the central arch, so that the requisite piers may be nearer the margins, than to make a series of arches of equal or nearly equal size, and of similar form.[7] The latter, however, is the usual course; and the noblest rivers are thus divided into narrow rapids, and, with few exceptions, a dull and tame monotony pervades the architectural composition of bridges from one end of the world to the other.

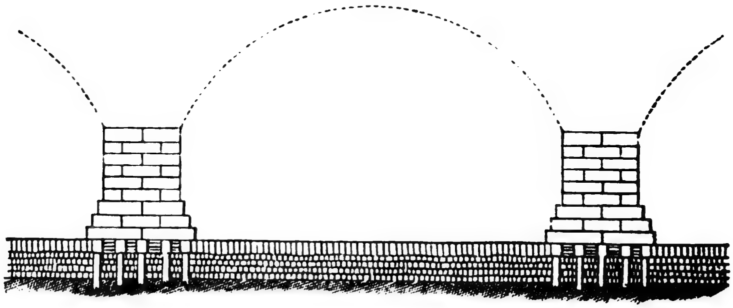
Abutments to bridges of masonry or of brick-work, or of a combination of the two, should be designed in such manner and with such effect, by means of back buttress, or counterfort, and wing-walls extending longitudinally inland and parallel to the direction of the thrust of the bridge,—that is, in the absence of such natural backing to the abutments as may be in itself immoveable, and thereby equivalent to constructions,—that they may be perfectly competent to the duty of sustaining the bridge, or of resisting the thrust of its arch or arches without the aid of the earth or other material of which the approaches may be embanked; and in the same manner the bridge abutments should have mass enough to hold

[7] Mr. Brunel has lately built a bridge to carry the Great Western Railway over the Avon at Bristol, exhibiting in a very agreeable manner an arrangement of the kind suggested in the text; the central arch spans 100 feet, and two collateral arches 30 feet each.

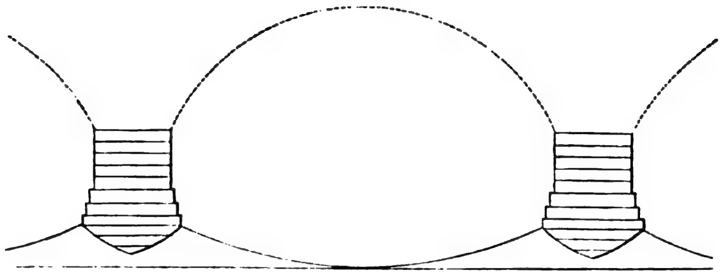
up any embanked approach without liability to be forced forward upon the arches which they are intended to sustain. Road or other bridges, however, upon dry ground, or where the space to be borne over can be laid dry, occurring within an embankment to carry another road, a railway, or a canal, may be designed as if they were culverts to receive and sustain external pressure, an inverted arch being turned from side to side under the bridge to hold the sides or abutments up against the thrust of the ground; but in such a construction counterfort and spandrel walls must be carried up in connexion with the constructions to make the culvert bridge independent of the earth-work formations, and able to stand alone, and to bear without assistance whatever loads may have to pass over it. Most failures in such works arise from the constructions not being made independent of the earth-work formations. Abutments cannot, indeed, be made too strong, but the strength necessary in any particular case must depend upon the form and extent of the arch or arches of which they have to sustain the weight and thrust, upon the weight and density of the materials used in them, and upon the ground upon which they may rest. When the arches are flat, and the thrust is consequently severe, the abutments should be benched back to prevent them from slipping on their platforms or other bases, but the benches must be upon ground strong enough, or the ground must be made, by skew-piling or otherwise, strong enough, to resist any lateral pressure that may be communicated through the abutments.

The inverted arch furnishes the most efficient means of extending the base or bearing surface of a bridge; and when circumstances render it necessary to protect the bed of a river over which such a structure is to be raised from the action of the current, this end may be answered, and the bearing surface extended to the greatest degree that can be required, by the use of a flat inverted arch. That the inverted arch in such a case may not be undermined, however, it must be flanked and protected by longitudinal grooved or dovetailed sheet-piling driven as deep as the ground can be disturbed, and longitudinal beams as fender-sills, to which the heads of the sheet-piling should be attached; but as the fenders, to be efficient, should be in long straight lengths, to which the curved sides of an arch will not adapt themselves, the width of the invert at its springings may be limited to that of the upper arch, and groining will give the means of extending it laterally, until a straight line of flank is produced on the level of the lowest plane, against which the sheeted fender-sills may be laid with the best effect.[8] A mere flooring of masonry flanked with sheet-pilings, as practised by that most ingenuous as well as ingenious bridge-builder, Semple, at Dublin, in the Essex and Ormond bridges, would answer the purpose of protecting the bed of the river; but a mere floor does not necessarily extend the bearing surface under the piers, and in Semple's examples it does not appear but that piling under the piers might have been

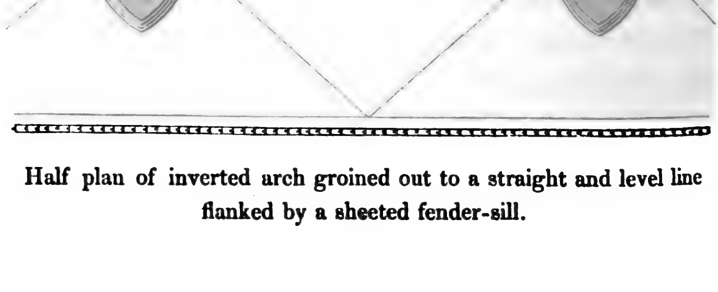
[8] See the second and third diagrams on the next page.



Floored bay of a bridge, with piling, &c., under the piers, according to Semple.



Elevation of a bay as last, with invert groined out to a straight line, as shown in plan below.



Half plan of inverted arch groined out to a straight and level line flanked by a sheeted fender-sill.

altogether dispensed with, if he had laid his flooring as inverted arches to extend the bearing surface, even without groining out to the dovetailed sheet-piling and fender-sills, as above described, and as the latter diagrams indicate.

These remarks are upon the supposition that the sites of two piers, at the least, and the space between them, can be laid dry at a time, whether by turning the course of the river or by including such an extent of surface as that indicated within one coffer-dam. Semple made a coffer-dam over half the whole breadth of the Liffey, and thus the head of the dam, or that part of it which ranged through the central bay or archway in the direction of the stream, served for both sides, the canted returns to the banks alone requiring to be reversed. He does not state, however, in what manner he completed the floor of the central bay where the head of the dam stood in the way, though the obvious mode seems to be to make it up to the piles of the dam on each side, and upon the removal of the same to cut off the piles in this part, and to complete the masonry bed or floor between the piles by the aid of a diving-bell, leaving the heads of the piles to form a part of the floor. It is true that an inverted arch could not be so well treated in that manner, nor can inverted arches be contemplated when the bays are of great magnitude; though in a design of the kind before suggested of a very large arch with two small arches next the abutments, these latter might have inverted arches with great advantage to the construction, as the bearing of the piers is thus distributed over a

large surface, and the thrust of the central arch is carried more effectually up to the abutments. All solid resisting matter placed in the bed of a river, however, whether as inverted arches, or as spreading courses or footings under piers, with whatever sheet or other piling may be requisite to enclose or defend the constructions, should be placed, not only below the bed of the river, but so much below the ordinary or hitherto bed, where no obstructions having the effect of piers may have existed, as to allow the channel to be deepened sufficiently to make the section of the water-way in the bays equal to the whole section of the bed or trough occupied by water before the erection of the piers, without bringing any part of such substructions and thorough works within the action of the current. Indeed, it is not undeserving of consideration whether the design of a bridge, whose works will have the effect of narrowing the water-way, should not always contain a provision for deepening the channel to such an extent as that here contemplated. It is true that under ordinary circumstances the current will effect such deepening of itself, but while it is in progress in this manner, the bridge piers are exposed to a greater wearing action than is necessary; and if, as in the case of Smeaton's Hexham Bridge, the ordinary force of the current is not strong enough to scour the bed of the river in the bays to obtain the space in section that the waters at some times require, when these occasions occur the bridge may be undermined by the violence of the action the bridge itself tends to produce; whereas the restoration of the space taken up by the piers and

abutments to the extent of their section would leave the water-course in that respect as large as the waters require, and in all probability no further action would be induced. But in the Hexham Bridge case no such thing was done, but the converse was effected. Mr. Smeaton knew the stratum of gravel upon which the bridge was built to be thin, and that it rested upon a loose sand which would yield to the action of the water if once exposed to it. He therefore placed his bridge upon the gravel, and endeavoured to prevent the water in floods from acting upon it "by the deposition of some matter more compact than the gravel itself;" but it is evident that this additional matter, of the excellence of which as a defence Mr. Smeaton speaks highly, still more diminished the already contracted water-way,[9] and tended to raise the flood-waters to the head requisite to scour it away with the gravel under it, by which means the sand was exposed, and of necessity the piers were then at once undermined, and the structure fell. If Mr. Smeaton had taken the view here propounded, that a water-way should have been preserved between the piers and under the bridge equal in sectional area to that which the unobstructed channel afforded it, or rather, perhaps, to the channel the water had made for itself, he could not have calculated upon the stratum of gravel remaining as a bed for his work, and this, his only failure, would not have occurred. It is strange, indeed, that the forewarnings which floods gave during the progress of the work, had

[9] See diagram, page 126.

not induced him to take steps to increase the water-way rather than still further to diminish it. He had been advised by the superintendent at the works of the effect floods were producing upon the bed of the river in the bays, as it appears from the following extract of instructions as to piling round the piers, and placing defences of rubble. He says, "As it appears from all the arches' interspaces wearing deeper, that it is the natural effect of the waters being more confined than at first by the interposition of the piers, it therefore indicates that we should not fill up more than what is absolutely necessary for our security; for the more we block up, the more tendency the water will have to take away the blocking and deepen the interspaces."

While upon this subject, it may not be out of place to remark upon the reason which Mr. Smeaton gave for the failure of Hexham Bridge, as he appears to have overlooked the fact, notwithstanding the intimation given of it in the foregoing extract, that the bridge itself gave rise to the action to which he refers the accident. "Reflecting,"—he says, in his report to Mr. Errington, the gentleman at whose risk the bridge had been built,—"reflecting upon every circumstance that has yet been communicated to me, with all the precision I am able, I am of opinion that the true cause of any failure was occasioned, not only by the great violence wherewith the bridge was attacked, but by the great weakness of the stratum of matter that lies immediately under the bed of the river, and which has been said universally to prevail in that neighbourhood by those

who made trial thereof, between the building of the first bridge and that of the second; which weakness of the under stratum I was not only aware of, but turned my thoughts towards every expedient that could tend to avert the ill effects that might arise therefrom. And having observed that, in all the attempts of those who had gone before me in this enterprise, they had dug considerably into the bed of the river, and thereby rendered that weaker which was already too weak, I did not doubt but that, by a contrary practice, my endeavours would have been crowned with the wished and expected success; for, as I had read of buildings and bridges that had stood upon more weak natural foundations than this appeared to be, and even myself had a case of the kind that I had effectually remedied, I did not doubt, but that with the precaution of not weakening the upper crust of hard gravel, but building immediately upon it, I should in like manner succeed in this place. The instances, however, that had come to my knowledge, though the strata under the foundations might be naturally weaker, yet none of them are liable to be attacked with any thing near that degree of violence that this river now appears to be capable of. Had it been possible for me to have been acquainted beforehand that a flood of this river could come down with so much suddenness, as that, for want of time for the lower reaches of the river to be filled from the upper, there could be created a fall or difference of level between the up-stream end and the down-stream

salient point of the same pillar, of no less than five feet perpendicular, which would in effect create a velocity of the water of above a thousand feet in a minute; —I say, could I have been informed of this single fact, as appeared to be at, and for some time before any degree of derangement was apparent in this bridge, I never could have thought of advising you, or any private gentleman, to have undertaken, at his own risk, a building of so much danger and hazard: and, exclusively of that danger and derangement which might naturally be expected to arise from the mere rapidity of the water, I am further of opinion from what now appears, that the mere difference of the weight of the body of water immediately above the bridge, which could not be counterbalanced by a body of water of an equal breadth immediately below, has, in reality, been sufficient to force down the under soft stratum out of its former position, so as to be more inclined to the west, and occasion the upper stratum, upon which the bridge immediately stood, to follow it."

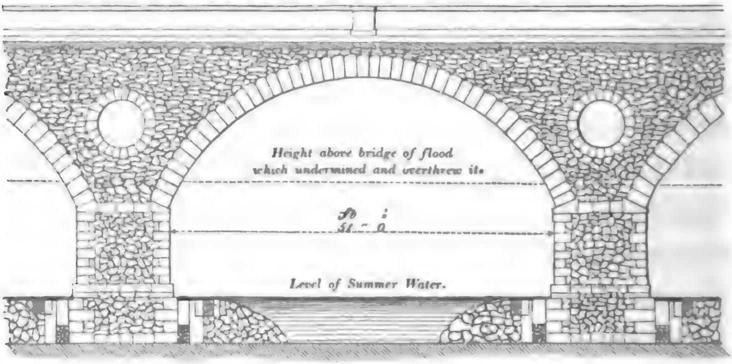
Hence it appears that Mr. Smeaton had no reason to suppose that the bed of the river in the part where he built the bridge was liable to serious disturbance from the violence or rapidity of the waters in the worst floods, as he attributes the destruction of a former bridge near the same place to the weakening of the stratum of gravel which formed the bed of the river by digging into it to found the piers, and not to any liability of the "upper crust of hard gravel"

to disturbance from the ordinary action of the floods to which the river had been always subject.[10] What may be the rapidity of sudden flood-waters over the bed of the river Tyne abreast of Hexham does not appear, but it is quite clear that it had not been sufficient to disturb the bed of the river where the bridge stood, or a head of five feet of water would not have been necessary to scour it out from under the bridge. Indeed, there is no reason to suppose that the crust of gravel upon which the bridge was placed would have been broken up by the flood in which the bridge fell, if the bridge had not been in the way to form the head and the consequent deep and rapid fall that ploughed it up. It was the bridge and the works connected with it that prevented the lower reaches of the river from being filled fast enough to prevent the suddenness of the flood from raising the head, or "the difference of level between the up-stream end and the down-stream salient point of the same pillar."

The river Tyne, in the part where the bridge stood, was about 530 or 540 feet wide at the height of freshes or ordinary floods; this width the piers and abutments reduced one-fifth, leaving but 424 feet opening between the piers where they are thinnest, and the diminished width was further reduced below summer water level

[10] Mr. Mylne, who was called upon by the Magistrates for the county to advise them after the disaster to Mr. Smeaton's bridge, says, in his Report, "that the bed of the River Tyne seems to shift and alter its form, extent, and situation, with every flood, more or less;" but upon this point, as well as upon several others, Mr. Smeaton and Mr. Mylne appear to have differed materially.

by the greater substance of the piers at their footings, and by the defences of rubble packed around them which had the effect of making an imperfect dam or sunken weir at every archway. But the flood-waters had on previous occasions risen above bridge to the height of the springings of the arches without injury to the works, notwithstanding the pressure of a head of three feet of water accumulated by the obstructions of the piers and



Longitudinal section of the central arch and its piers, of Smeaton's Hexham Bridge, showing the rubble defences to the piers and their effects upon the water-way.

their defences, and a scour arising from a velocity of from 800 to 900 feet per minute resulting from that head. Above the level of the springings, however, the arches themselves began to be immersed, and these in a rise of three feet in height above the springings will have further diminished the water-way full thirty feet, when the pressure became sufficient to break up the crust of gravel that lay exposed between the toes of the rubble defences of the piers, to wash out the sand and loam from between the rubble, and to scour out a channel deep

enough to compensate for the space that the piers of the bridge with their defences and the immersed haunches of the arches occupied. Had this last operation been considered and treated as an essential part of the design of the bridge, or had Mr. Smeaton contemplated the propriety, not to say necessity, of doing so, either the bridge would not have been built, or it would have been designed in such manner as not to leave it the certain agent of its own destruction; and yet Mr. Smeaton concludes a letter detailing the circumstances of the catastrophe, three months after it occurred, with the remarkable expression, "all the experience I have gained is, not to attempt a bridge upon a gravel bottom in a river subject to such violent rapidity"!

It seems quite evident in this case that the bridge was the cause of its own overthrow, and it is very likely that the yielding of the bed of the river prevented the greater calamity which must have followed if this had not happened; the waters must have risen until the banks above should give way or be overflowed, when the whole country would have been deluged to the probable destruction of life and property beyond calculation.

A further illustration of the view here taken, that the velocity to which Mr. Smeaton referred the accident to Hexham Bridge was not attributable to any thing peculiar to the river, rapid as such a flood in it would be, is derivable from the circumstance before alluded to (p. 58, *et seq.*) connected with Old London Bridge. The Thames is not subject to the sudden bursting of waters and the fearful velocity of current which Mr. Smeaton

charges the Tyne with, and yet a rapid was formed under the central arch of the old bridge that threatened the downfall of the structure, by exactly the same process by which the Tyne destroyed Mr. Smeaton's bridge,—by scouring the bed of the river, and undermining the constructions that dammed its waters up to a head sufficient to produce the injurious effect, although no treacherous stratum of loose sand lay immediately under the ground upon which the piers of the old bridge stood.

The opinions and practice of Smeaton must be at all times received and considered with respect, and not the less so that he did not always recognise the real causes of the actions with which he had to contend, but he stated his views with so much candour and clearness as to lead the inquirer to a free consideration of their correctness. It is remarkable, however, that the man who in designing the Eddystone Lighthouse acted fully up to the only sound practice in hydraulic architecture,—the entire exclusion of water from the constructions and formations that are to be opposed to the action of water,—and thereby perfectly succeeded in establishing a work capable of resisting the utmost violence with which unrestrained water can act, should hope to effect any permanent object by the deposition of uncombined rubble in water to counteract the effect of water liable to be put into motion with any degree of power, as at London Bridge,—or as at Hexham by means of such a defence as a heap of shapeless stones bedded in loam to protect the bases of piers laid in cassoons upon the surface of a stratum of gravel from the effect of the certain per-

colation of the water between the floor of the cassoon and the gravel upon which it was placed. It may be taken for granted, that where water can find access it will make a passage, if there be but head to give pressure and induce a current: in the case of Hexham Bridge, the constructions themselves acted to form the head, and the heaped rubble could not prevent the water from free access under it to, and under, the imperfectly bedded cassoons; whilst the simple law of nature which deprives a stone surrounded with water of a large proportion of its gravitating force, left the defences themselves at the mercy of the current as soon as a head had formed sufficient to wash out the loam and overcome the remaining force of gravitation of the separate and immersed pieces of stone. If the rubble that was packed around the piers of Hexham Bridge had been laid with good hydraulic mortar in a sound and impervious stratum under the masonry of the piers, below the bed of the river, though such layer had been coffered in the gravel and bedded on the sand, the bridge would have had another chance of permanence in the absence of the partial weir the defences formed, and in the greater depth of the foundations; though indeed the diminution of the water-way by the piers, and the lowness of the springings of the arches, might have been sufficient for its destruction; or, as before remarked, the bridge might have been preserved for the devastation of the surrounding country. The instructions given by Mr. Smeaton from time to time during the progress of the works of Hexham Bridge, and particularly those having

reference to the restoration of parts which were injured by floods while the bridge was in course of construction, show with sufficient clearness, nevertheless, that he had kept the exclusion of water from the rubble defences in view, though the measures he took to that effect were not adequate to the duty. Indeed, it seems very likely that Smeaton undertook the construction of a bridge at Hexham with the intention of succeeding where others had failed, without greatly exceeding them in expense; whilst the course he pursued to attain success really tended to insure his failure. Mr. Smeaton proposed, by increasing the arches from seven, the number the preceding bridges had had, to nine, to distribute the weight more equably over the bed of the river, so that, to use his own expression, " the bridge might have more legs to stand upon, in consequence of the natural weakness of the stratum;" but he seems to have overlooked the effect of the solid " legs," and of the defences to their feet, in holding up the water more certainly, and so to counterbalance the benefit he counted upon deriving from them.[11]

The substance in thickness of which bridge piers may be built must depend, in a great degree, upon the materials of which they are composed, the height to which

[11] Even Mr. Smeaton might have read with advantage the following passage from Belidor's *Architecture Hydraulique:* " Rien de si dangereux que de resserrer les eaux courantes, à cause de leur renflement du côté d'amont, qui causeroit une cataracte difficile et même dangereuse pour le passage des bateaux, et qui pourroit occasioner des affouillements capables de dégrader le pied des piles et des culées." Part II. liv. 4, p. 442.

it may be necessary to carry them, and the weight of the arches, upper works, and load; it being taken for granted that the workmanship in execution will be such as to render it unnecessary to make any allowance in that respect; for no judgment in the design will avail against bad work, as the thickest piers, badly wrought and built, may be unable to bear the weight they are intended to carry, though piers of half their substance might be sufficient.[12] Nor should it be necessary to have reference to the extent of base which the nature of the ground may appear to render necessary, as means should be adopted of extending the footings, and otherwise spreading the bearings of piers upon the ground, or upon an artificial stratum within the ground, so as to render them altogether independent in that respect, even if their substance be diminished to the greatest possible degree of thinness.

It was the practice of the early builders of arched bridges to consider every pier in a bridge as an abutment to the arch resting upon it; but as the desirableness of retaining the greatest amount of water-way and head-way under a bridge, with easy rise to and upon it, became

[12] If the piers of Westminster and Blackfriars' bridges had been one-eighth or one-ninth the span of the arches resting upon them, instead of one-fourth and one-fifth of that proportion, as they are respectively, it is not improbable that both these bridges would have failed: the late operations for the repairs to their piers have exposed workmanship of the worst kind; even in the outside or ashlar courses, stone chips and pieces of slate, and even deal chips, were commonly found packed and wedged in to compensate for the leanness of the stones in both beds and joints.

recognized, flat arches,—segments of circles less than semicircles, and semi-ellipses,—came into use: but as these exercise a greater lateral thrust than the semicircular or high pointed arch, (one or the other form, or a near approach to the former, having been almost universal,) it was necessary either to cease to require of piers to act as abutments under any circumstances, or to give them power of resistance enough to perform the duty. To do this latter would be to sacrifice the more important water-way in the transverse section for the less important in the vertical section. The pier was therefore allowed in many cases to retain, with the flat arch, the same proportion to the bay or span of the arch that custom had accorded to it with the deeper and less thrusting semicircular and high pointed arch; and in many instances it was reduced below that proportion, whilst the demand was still made upon it to perform the duty of an abutment, temporarily at least, or until the thrust of the first constructed of the two arches bearing upon it should be balanced by the completion of the second. The main object contemplated appears to be, that subsidence or other injury to any one pier should not be felt beyond the two arches resting upon it at the utmost; and it is further claimed for such an arrangement, that it has the economical advantage of leaving the centering more at liberty, as the centering of an arch whose piers are strong enough to resist its thrust, may be removed to serve for a second, and so on throughout the whole extent of a bridge of equal arches. Such advantages are, however, but poor compensation for the

injury done to a river for the purposes of navigation, and to the country bordering a river, by a bridge which dams the waters until they form a rapid; and it may be truly asserted, that more bridges have been ruined by the effect thus produced by themselves, than have suffered severe injury from all other causes of destruction together, bad workmanship not excepted. Perronet, a name of great and deserved respect in all matters relating to bridge building, speaking of the practice of dividing a bridge of considerable length into several independent parts by large or abutment piers, as applied in the bridge of eleven unequal arches over the Loire at Blois, by Gabriel, one of his predecessors in the office of chief engineer of bridges and highways, makes the following suggestive observations: "I think that it may be prudent in designing bridges for rivers of great width to introduce some strong piers, which in case of need may serve as abutments, placing them at distances of three or four arches apart;"—the eleven-arched bridge at Blois has two such piers, at four bays from each abutment, and with three bays between them;—"such an arrangement will afford, moreover, the means of constructing large or extensive bridges in different parts successively, of which each part may be considered as a complete bridge, having its own independent abutments; but strict care should be taken not to contract the beds of rivers by introducing these thick piers within them unnecessarily."

"Many examples may be cited," the same author continues, "of mischief that has resulted from inat-

tention to this consideration in bridges of the best construction; but I shall content myself with referring to one well-known case.

"In the bridge of three arches of 105 and 138 (French) feet span, built over the river Allier at Moulins, upon the design of Hardoin Mansard, the piers were 32 feet in thickness, and might have served as abutments to the two smaller or land arches at least. This bridge was thrown down, nevertheless, in 1710, immediately after its completion, and the accident has been attributed to the contraction of the water-way by the bulky piers: the bed of the river is composed, however, of a fine sand, which the current, during floods, early disturbs and carries along with it."[13]

Another bridge was built in the place of Mansard's, fifty years afterwards, by M. de Regemorte the younger, who substituted for the bridge of three large and lofty arches upon piers occupying less than one-sixth the space between the abutments, a greatly lengthened bridge of thirteen low and equal semi-elliptical arches upon piers occupying one-seventh of the whole space between the abutments, taken at the level of the springings of the arches, which is that of summer water; but as these low arches gather over quickly, the flood-waters are contracted as they rise, and De Regemorte's bridge was saved, in a flood which occurred in 1790, only by the rupture of one of the embankments leading to it.

"The piers of bridges ought to be considered," says Perronet in the commencement of the Memoir above

[13] Œuvres de M. Perronet, pp. 624, 625, 4to. Paris, 1788.

quoted, "either as performing the duty of abutments, or as relieved of that duty by the counteraction of the collateral arches, through which the thrust is carried from abutment to abutment of the bridge. In the first case, piers should be rendered as capable as the abutments themselves ought to be, of resisting lateral pressure, that they may withstand the lateral thrust of the arch-stones which tends to overturn them, and which increases by so much the more as the arches are flatter and the piers loftier. In the second case it should be enough to give the piers [14] substance to enable them to carry the weight of the two half arches which are raised upon the two sides of each pier respectively," together with so much of the upper works as lie over, and as may be brought by means of the arches to bear upon each pier, and the load the bridge may be destined or become liable to receive;—it should be added,—though it may, perhaps, be fairly understood from what is expressed.

The bridges, and ruins of bridges, which remain to the present day, of Roman construction, exhibit for the most part great massiveness in the piers, as these are in some cases equal in thickness to one-half the spaces between them, in some to one-third, and in few instances do the piers bear less than the latter proportion to the

[14] The term pier, used in a general sense, means the whole of the masonry below the springing of the arches; but as used in the text at the point noted, the smallest part, or what in a pedestal would be distinguished as the die or dado, is intended. This is distinguished in French by the term *pied-droit*, and Perronet, in using it, explains that the vertical (*à-plomb*) part of a pier, or from the top of the footings to the springing of the arches, is so called.

openings. The bridges of the middle ages, such as those over the Elbe at Dresden, and over the Rhone at and below Lyons, have piers or masses of substruction to the piers occupying almost, if not quite, as large a proportion of the bed of the river as they leave to the water; and to these and such like nuisances, inundations,—of which the fearful inundation at Lyons in the winter of 1839-40 is an instance,—may be certainly traced, whilst the bad drainage of the lands bordering upon rivers vexed with bad bridges is a constant source of evil to the countries which possess them.[15]

In the beginning of the sixteenth century some diminution took place in the relative proportion of the piers of bridges to the span of the arches; and one-fourth the opening or one-fifth the space occupied by a pier and two half arches may be considered the average practice from that period until the time of Gabriel, the author of the bridge at Blois, as before mentioned, early in the eighteenth century; but during the lapse of two centuries the semi-elliptical arch had been brought into very general use, and flatter segmental arches were built than had been hitherto practised, involving an abandonment of the principle which required the piers to be equal to the duty of abutments, since the piers practically became slighter as the arches were made flatter; and although the water-way derived no benefit from the change, except as it led to arches of increased span and consequently

[15] The removal of Old London Bridge, which was a bridge of the class alluded to, has tended materially to improve the drainage, and consequently the healthiness, of all the low parts of London above bridge.

reduced number, the road-way was greatly improved where the depressed arch was applied. A distinguished case of the kind of composition here referred to is the celebrated Florentine Bridge before cited, that of the Most Holy Trinity, built between the years 1566 and 1569, the piers of which might almost serve as abutments, not only, as Perronet remarks of Mansard's bridge at Moulins, to the two smaller or land arches, but to the central arch itself, flat as the arches are, and severe as their thrust must be. The substance of each pier of the Trinità, as the bridge is commonly designated, is equal to more than one-fourth the central bay, and to above two-sevenths of one of the smaller bays, whilst the whole space occupied by the piers bears nearly the same proportion to the whole width between the abutments of the bridge that Mansard gave more than a century later to his bridge at Moulins. This latter work would appear, indeed, to have been arranged with reference to Ammanati's bridge, except as to the rise of the arches, which was very much greater in Mansard's than in the Trinità. As a great part of the difference in rise of the arches in the two bridges is given by Ammanati to the water-way, by placing the springings of his arches at flood-water level, whilst Mansard's arches appear to have been sprung at the level of lowest or summer water, in all probability the endurance of the one and the destruction of the other may be attributed in a great degree to these circumstances.

As the eighteenth century advanced, a further diminution was made in the relative proportion of piers to the

openings of the arches they supported, and one-fourth became the minimum, and one-fifth the usual practice; that is to say, the substance or thickness of piers was seldom made more than equal to one-fifth the space occupied by a pier and two half arches, whilst it was reduced in most cases to a sixth of the same space, and in many to much less than that proportion. M. Hupeau, Perronet's immediate predecessor as chief engineer of bridges and highways, left at his death an unfinished bridge of three arches over an arm of the Seine at Mantes, in the road from Paris to Rouen, to be completed under the direction of his successor. This had been designed with piers equal in thickness to two-ninths the openings of the smaller arches, to which they were to serve as abutments while the centering was transferred from one of the arches to the other, and pending the construction of the central arch. For this duty, however, the piers were found incompetent, or rather they were found incompetent to withstand the thrust of the centering with the weight of the unkeyed arch upon it, as the pier that failed slipped upon the piled platform on which it was built while the arch was still incomplete, and, therefore, before it could have exercised any thrust upon the pier but through the centering upon which the weight of the stones of the incomplete structure rested. This step-child of M. Perronet, it may be further remarked, is designed upon a most vicious principle; for what in it are considered as piers, are not such in reality, but are mere footings, from the top of which, and *below the level of low water,*

the arches are sprung. Hence the water-way is at the best of times narrower than the thickness nominally given to the piers would indicate, whilst the appearance of the work and its condition, both as it regards the navigation and the passage of flood-waters, are alike impaired thereby, putting out of the question, for the moment, the stability of the bridge, and its effect upon the river as a dam to the stream in floods. The proportion of the pier to the opening in M. Hupeau's great work over the Loire at Orleans is somewhat less than one-fourth the span throughout, but as at Mantes, though not in the same degree, the nominal substance of the pier or its thickness at the springing of the arch gives but a deceptive idea of the water-way occupied by the bridge constructions, since the pier begins to spread where the arch springs, and consequently the contraction of the water-way increases with every course both above and below the line of springing.

In Westminster Bridge, which is of rather earlier date than the two last-mentioned bridges, the substance of a pier below low-water level is one-fifth of the space occupied by the pier itself, and by the two unequal half arches resting upon it, or truly one-fourth the span of an arch, though not exactly in that proportion to any particular arch-way. The semicircular arch used in this bridge abstracts less of the water-way than a semi-elliptical arch of the same span and sprung at the same level would have done, but even with that advantage the width of water-way is nearly fifty feet, or about one-sixteenth, less at the height of ordinary spring tides than

at the level of low water in the river; so that the piers are thus rendered of greater thickness in mischievous effect than they are in real substance.

The piers of Blackfriars' Bridge occupy one-sixth of the space allotted to a pier and two half arches, or are themselves equal to one-fifth the span of an arch; but this is only between half-tide level, at which the arches are sprung, and the level of low water, below which latter the piers spread and consequently occupy a greater proportion of the bed of the river than they do between the low-water level and their termination at the springing of the arches.

Smeaton's bridges stand for the most part upon piers taking up about one-fifth the space occupied by a pier and by two half arches, or one-fourth the span of an arch, and the piers have generally height in the body (die or dado, *pied-droit*) to raise the springings of the arches considerably above the ordinary level of the water in the rivers they cross, though not, it appears by the instance at Hexham, so high but that floods with the aid of the piers themselves could reach them. Mr. Smeaton's estimate for his bridge over the Tay at Perth contemplates the removal of the centre complete from arch to arch, and provides booms to strut the piers one from another to hold up the arches pending the removal of the centre from bay to bay, so that the piers were looked upon as competent to act as abutments with such slight assistance as the provided strutting would afford.

Gautier, early in the last century, propounded the

principle that should determine the capacity of bridge piers, though it remained to Perronet to carry it out in practice. "There can be no doubt," said Gautier in 1716, "that the piers of bridges support only half the masonry of the two arches at their sides, taking the arches from the middle of their key-stones." [16] Perronet does not acknowledge any obligation to Gautier, although in the Paper before referred to,[17] which was read before the Royal Academy of Sciences at Paris in 1777, he re-states the principle, and adduces in support of it the argument to be drawn from the weight of materials in construction borne by piers in churches both ancient and modern, not in words certainly, but to exactly the same effect that both are found stated in Gautier's earlier work. Not contented, however, with the arguments supplied to him by his predecessor, Perronet attempts to justify what he had then, in a limited degree, already practised in the bridge at Neuilly, by reference to works in which the piers had been made too slight to serve for abutments to the arches resting upon them, though, indeed, his best practical illustration of the truth that piers need not occupy a fifth, a sixth, or even a seventh part of the water-way over which a bridge is to be built, is that which Gautier had pointed out. Perronet certainly went further in the application of the principle than Gautier appears to have imagined to be within the bounds of prudence in bridge building; but there is no sufficient reason for stopping in the reduction of the relative thickness of the

[16] Traité des Ponts, p. 103, 8vo. Paris, 1716. [17] Page 134.

piers to the span of the arches even where Perronet stopped:—"The information I had acquired," he says in his description of the Neuilly Bridge, "of the capability of the stone[18] to support the weight with which it might be loaded induced me to believe that the thickness generally given to piers, and which is commonly estimated at one-fifth the opening of the arches, might be greatly reduced: such estimate would have given twenty-four feet to the piers of the Bridge of Neuilly, but I have contented myself with giving them thirteen feet, though in strictness this might be reduced to ten feet, or twice the length, or depth in vertical section, of the key-stones of the arches, which proportion may be regarded as the minimum for the thickness of bridge piers, but quite sufficient, nevertheless, if needful precautions be taken to insure the soundness of the construction."

The gauge thus established for the thickness of bridge

[18] Meaning the particular stone employed, of which the writer says in a note to the Memoir read before the Academy of Sciences,—"It had been determined from experiments made by M. Soufflot, and repeated by myself, that to crush a square foot would require a weight of 240,000 lbs., or a column of equal base 1580 feet high,—a cubic foot of the stone weighing 152 lbs. My own calculations showed me that in similar proportions a pier of the new bridge was not loaded with more than equivalent to a column 121 feet high, so that the piers are strong enough, or have substance sufficient, to enable them to carry twelve times the load they have to bear."—The general utility of the foregoing information is not affected by the circumstance that the weights and dimensions are not exactly those understood by the same terms in England, as the available information is in the proportions, and not in the weights and dimensions.

piers,—the length or depth of the key-stones of the arches,—may be ascertained by a calculation of the weight the key-stone may be required to resist, which is easily determinable, as it is merely that of half the materials in and over the arch, surplus being always allowed for the load and for contingencies to such an extent as to render failure, from a near approach to even the minimum power of endurance of the material employed, impossible. Experience has shown, in many instances, that the substance of a pier for the purpose of sustaining the weight of vaulting may be brought greatly within the relative proportion in area of the vaults sustained, to the sustaining pier, which the Neuilly Bridge exhibits. The chapter-houses of many of our cathedrals furnish striking examples of the comparatively great extent of stone vaulting that may be safely imposed upon the same stone in well-built piers. The central column of the chapter-house at Wells bears a hundred superficial feet of vaulting for every superficial foot of its own area in horizontal section; whereas the piers of the Neuilly Bridge, greatly as they are reduced below the former practice, give *ten* feet of bearing surface in their horizontal section at the springings of the inner arches, to a hundred feet superficial of the vaults they sustain, or one to ten instead of one to a hundred. This consideration may tend to give confidence in adopting a much lower scale of proportion of bearing substance in piers, to the surface or extent of the vaultings and upper works to be carried; for although it is quite certain that the material constituting the

pier is more liable to have its surface eroded, and otherwise injured, when placed in a water-way, than when within the walls of a cathedral chapter-house, a thin pier in a water-way is subjected to less violent action from the water than a thick one, because it restrains the water less, and it is less exposed to casual injuries, because it occupies less of the water-way, and is therefore less in the way of injury. It is in comparison no objection to a thin pier,—if it have thickness enough to receive the beds of the springing stones of the arches on either side of it,—that it is liable to be rendered over weak by the degradation of its faces, for the arches rest as much upon the outer or surface portions of a thick pier as upon the two halves respectively of a thin one, and arches would fail through the degradation of the surfaces of thick piers as readily as through the destruction by erosion or other external injuries of thin ones, if in either case the degradation were allowed to go on without restoration, and restoration of the one may be effected as easily as restoration of the other. Indeed, the practice in constructing the bulky piers of bridges in past times was to heart them up, or fill in between the wrought and gauged masonry of the external faces with rough rubble, as indicated in the diagram at page 126, *ante*, or at best to build with inferior coursed work through the heart of a pier. In such cases the arches and upper works must depend entirely upon the closely-wrought masonry of the faces of the piers, as the different sorts of masonry have no common bearing, and in any such cases the piers might,

with the same degree of security at the least, have been restricted to the wrought masonry of the faces brought close and bonded together, with the slight increase in their substance that the increased weight of the arches from their increased extent would require. Of this remark the section of a pier and half arch of the Wellesley Bridge at Limerick, as shown in Plate 55, affords an apt and clear illustration: it is quite evident that the coursed rubble hearting to the pier carries nothing but the equally useless pile of similar work immediately over it, and that the springing stones of the arches and the gauged masonry under them might have been brought up back to back and bonded together with great advantage to the construction and great relief to the water-way, for—it cannot be too often repeated—a bridge pier in a water-way is at best but a necessary evil, and the thicker the pier the greater the evil.

In reducing the thickness of piers to the smallest capacity consistent with sufficient strength to carry the load to be imposed upon them, although the substance of a pier should be independent of such considerations, it must not be overlooked or forgotten, nevertheless, that the piers do but sustain the weight to transfer it with their own, added to that of the arches and upper works, to the ground, where extent of base must be taken, or an artificial basis must be made within it, of sufficient extent to bear all without yielding, and the piers must be so arranged as to distribute the weight they bear and impose over all the surface requisite to sustain it. This may be done,—though only in certain

cases, as before intimated,—by turning inverted arches over the ground under the piers; but, in a large work, it must be done by spreading the footings gradually but sufficiently to cover space enough of a natural or prepared foundation to bear the load, whilst,—it has been already said,—the footings should be placed so low down as not to be within the action of the current. An instance of the practice here recommended occurs in Staines Bridge (Plate 53), where the artificial stratum of piling under the piers is so much below the bed of the river as to be obviously beyond the possibility of being acted upon by, or of acting injuriously upon, the current, and so much so, too, as to place the spreading courses or footings under the piers below the average depth of the water-way. An instance of the contrary practice in founding piers is exhibited in the bridge of the Champ de Mars at Paris (Pont de Jéna, Plate 56), where a great part of the piled substratum stands above the bed of the river, except on the shallower side, or where the bed lies highest, and even there the footings of the piers and abutments are entirely within the water-way. The obvious intention in this case is to compel the stream to act over the whole breadth of the bed by interposing the masses of piling and Smeatonian rubble defences[19] about the substructions on the right-hand side; but Hexham Bridge has sufficiently demonstrated the futility of attempts to restrain water by means of merely packed rubble when

[19] See *ante*, p. 126.

the water can be raised to a head; and the piled substructions, the spreading footings of the piers, and the rubble defences together, have the effect, in the bridge of the Champ de Mars, below the ordinary level of the water, of doubling the substance the piers exhibit above that level, and thereby contracting the water-way nearly one-fifth. In the two cases cited there can be no hesitation in giving the preference to Mr. Rennie's over M. Lamande's practice; indeed, if the transverse sections of the rivers respectively, as shown in the Plates, may be depended upon, Mr. Rennie would have been more justifiable in allowing his piling to stand higher because of the apparently even course of the current in the Thames where Staines Bridge stands; whereas the deeper water on the right-hand side of the elevation of the bridge over the Seine (being the left bank of the river) shows the bridge to be upon, or immediately below, a bend, where the stream acts with greatest effect upon the side on which it strikes, or upon which it is reflected. According to the representation which the engraving gives of the substructions of this bridge, (that of the Champ de Mars,) the right-hand abutments and the retaining wall of the towing-path might, under the violent action of a sudden flood, be denuded of rubble and left strutted upon the heads of the piles, and dependent upon the sheet-piling for the exclusion of water from among the ground piles. Supposing the piles to be driven as deep as they can be made to go, they should have been cut off at a level not above that of the lowest part of the bed of the river,

and at the same level right across, as in the case of Staines Bridge; and thus the footings of the left-hand abutment, and those of the two piers on that side, would have been below the bed of the river where they occur, whilst the solidly constructed work in the footings of the other abutment, with the towing-path wall and the other two piers, would have done all that should be attempted towards directing the action of the stream more equally over the whole breadth of the river.

One objection entertained to making piers so thin as to require considerable spread in the footings to obtain sufficient ground base, whether natural or artificial, is, that masons always work the beds most truly nearest the faces, or rather that they will not work the beds of stones as they should be worked but for a short distance within the faces, and that, therefore, footings should spread but little, that the superstructure may bear upon the well-wrought parts of the substructions, and not upon the lean and mortared beds lying in from the faces. Mr. Telford states this objection in the treatise upon the term 'Bridge' in the Edinburgh Encyclopædia; but surely it is most futile, and deserves no other consideration except as a warning to take care that no stone be set in a bridge pier or abutment, or in any arch of a bridge, that does not bear fully, fairly, and equably, over all. Masons will do what they are required to do and are paid for doing; but if a contractor have reason to believe that he may execute work according to the practice alluded to, he will tender to do the work for less money than if he were conscious

that a better practice would be enforced. Footings should not spread more than the strength of the stone employed will carry the weight imposed upon the top bed of a course, over the whole of its under bed; but this is the only limit that ought to be taken into consideration in spreading the footing courses to obtain a sufficiently extended base upon the substratum, whatever that may be.

It is true, nevertheless, that Perronet stands almost alone in the practice which he adopted of reducing the piers of stone bridges even to the limited and cautious extent, in proportion, to which he carried it in his great work the Bridge of Neuilly, and having reference to works of the same class, that is to say, to bridges the tangent of whose arches at the springing is at right angles to the chord, which can only occur when the chord is a bisection of a circle, or other regular curved figure, as an ellipsis or oval; for in works of which the arches are small segments of large circles, the proportion of piers to openings that Perronet professed to adopt at Neuilly has been repeated not unfrequently, and is found in the two examples last above referred to, Staines Bridge and the Bridge of the Champ de Mars, Plates 53 and 56. From the fact that the piers of Waterloo and London Bridges have piers equal in thickness to one-sixth the openings of the arches, or of the two half arches bearing upon them, whilst the piers of Staines Bridge, by the same architect, are only equal to two-seventeenths, or between an eighth and a ninth of the two half arches resting upon them,

it would appear that some idea of piers being in a condition, or nearly approximating a condition, to do the duty of abutments where the arches spring low down, as in Waterloo and London Bridges, pervaded their author's mind: with reference to Staines Bridge, and to bridges of the same class, such an idea would evidently be preposterous, and it is really not less so to treat or to consider piers of the relative capacity and of the height in the body,—that is, from the footings to the springing of the arches,—of those to Waterloo and London Bridges respectively, in any other light than as bearing pillars, for they could not resist for an hour the thrust of the arches upon them but for the counter thrust of the arches one to another carrying the thrust of every arch in the series over the heads of the piers up to the abutments. In this point of view the height of the pier from the top of the footings to the impost, or to the springing of the arch, is a matter of no inconsiderable importance; and it may be contended with good show of reason that the height which the rise and fall of the tide in the Thames at London renders it necessary to give in some form or other to the piers of the bridges upon it, places the comparatively bulky piers of Waterloo and London Bridges upon an equality with the nominally slender piers of the Bridge of Neuilly; for in truth the almost total absence of body (*pied-droit*) to the pier in the last-mentioned work between the high and rapidly spreading footings below, and the low level at which the inner arch is sprung above, leaves Perronet's prac-

tical reduction of thickness in the piers far less important and far less beneficial to the water-way than it pretends to be. It is only at and above the ordinary level of the water in the Seine that the reduction appears or exists, for below that level the footings begin to spread, and in two courses the pier has been increased in bulk to but little less than a fifth of the whole water-way. It must not be inferred, however, that no benefit is derivable from the reduction, even as Perronet has made it, for the footings spread no more, nor does the arch spring lower down because of the slightness of the pier at the springing, so that the water-way has all the advantage arising from the reduction, the other circumstances remaining the same; but the full extent of advantage derivable is denied, by the footings not being placed below the bed of the river, and in floods, by the lowness of the level, with reference to the level of the water in the river, at which the arches are sprung,—an objection common to almost all works of similar form and arrangement that have fallen under observation in the foregoing remarks.

That the level of the road upon a bridge may not be raised above the level of the roads which it connects more than is absolutely necessary, and that the head-way under a bridge may be as high and clear as it can be made consistently with the safety and utility of the road-way over it, it will be very generally necessary that the bridge itself should rise from the abutments to the middle, making the road-way on both sides inclined planes to and from the highest point.

When this is the case, the springing of the arch or arches at the abutments should be assumed at the level of ordinary high water, whether of floods or spring tides, and head-way for navigation, or for craft navigating the river, being taken under the middle of the bridge at the highest level the water attains, it will be readily determinable whether the inclination the road-way must take over the two assumed points is fairly practicable under the circumstances or according to the situation of the bridge and the facilities required of it.[20]

It is here taken for granted that the chord of the arch or arches shall be one and the same straight and horizontal line at or above the level of the highest ordinary water in the river the bridge is to cross, and higher if the water is liable at any time to rise high enough to endanger the proposed structure in the manner already shown,—so much higher, indeed, that the haunches of the arches can never be immersed. The crown of the bridge being taken, therefore, high enough to secure this effect, as well as to give head-way to navigation, and adapting again the proposed inclination of the road-way from it in both directions, the intervening space,—that is, the height between the level at which the chord or springing line of the arches may be taken, as above, and the line of the road-way,—is to be filled in with the works constituting the bridge. Where this space is very shallow, the arches must be flat, and whether they be

[20] See *ante*, page 88.

built of semi-elliptic form or in the form of segments of large circles may depend upon taste, though the question may, indeed, be determined independently of taste by reference to the practical question of the strength or power of resistance demanded by the one or the other form, of the abutments; or the space within which the arch must be brought may be so shallow as to require the merely cambered arch used by Perronet at St. Maixence. How far or under what circumstances the semi-elliptic or the segmental form may be preferable as a matter of taste will be discussed hereafter; the matter for consideration in this place is, the filling in with the best practical effect the most limited space in height between the level at which the arches may be sprung and that which the navigation requires above it under the crown of the arch or arches. As there need be no limit to the span of an arch but what the strength of the stone used, or its power of resisting pressure, imposes,—if it were not for economical considerations as to centering to hold up the mass while in progress of construction, and as to abutments to restrain or uphold it when constructed, and for considerations having regard to the form of the arch and the rise of the road-way,—in a large proportion of cases bridges might be limited to one arch, and in most instances bridges may be restricted to a much smaller number of arches than is usually given to them.

There is, indeed, another consideration that may interpose to limit the span of the arch or arches of a bridge, and that is, that the greater depth of arch-stone, or height in the key, which great span imposes, may operate in-

juriously upon the road-way by raising it higher than it otherwise need be carried. It may be more convenient to increase the number of arches in a bridge to reduce the length of the key and the consequent rise of the road-way, the head-way under the arch being supposed straitly limited, but it can seldom happen that this should be the better alternative, seeing that, by universal consent,—which will not be here disturbed,—every bridge should have an unequal number of arches, so that the number of arches in a bridge, when increased, must be increased two at the least, and two additional arches involve two additional piers, the expense of which may be much better thrown into the approaches of a bridge to raise them and the road-way upon them, and thus to obviate the disadvantages imposed by the great length of the key-stone of a large arch, than to make one arch three arches, three arches five, or to increase any greater number in like proportion.

The reconciliation of what are in most cases conflicting desiderata, the slightest rise in the arch or arches of a bridge with slight thrust upon the abutments, must be sought in curves for the arches which may have the effect of turning horizontal thrust into vertical pressure within the smallest space; but as strength in the abutments of a bridge is attainable at a comparatively easy rate, it is not worth while to count upon the efficient action in that respect of any curve or combination of curves to save in the abutments at the risk of failure of the whole structure. Let the abutments be so constructed that the materials of which they are built must

be crushed before they can yield,—taking care at the same time that the materials which receive the pressure shall possess the same power, in their own substance, of resisting pressure as the material may possess of which the thrusting and pressing arch is built. It is desirable, therefore, in constructing bridge abutments that may have to resist a very severe thrust, that resistance be not sought in the weight of the materials and in the holding or attachment of one stone to another through friction or through the adhesiveness induced by the presence of mortar; the constructive arrangement should be such as to oppose the material itself to the thrust.

The direction of the severest part of the thrust of an elliptical or other arch approaching the elliptical form, is in a line tangent to the flatter part where the curve begins to quicken, or where the upper curve may embrace about 60° of the circle that it most nearly approximates, and the courses of the abutments should be carried on at right angles to such line down to a firmly resisting natural or artificial uncoursed base: the same coursing may be made to embrace the backs of the stones of the lower curves, until the centre of gravity of the stones in the face of the arch fall within the vertical substructions of the abutments. The safest course to be pursued in abutting an arch whose form is that of a segment of a circle less than 60°, is to carry on the radiating courses to the extent of 60° of the circle, and then proceed as last indicated.[21]

[21] The practice recommended in this paragraph is the most certain for arches of great size, but it involves some disadvantages in con-

If a bridge be carried over a river with flat arches of great span, and the approaches require to be constructed at either end to carry the road-way over a flat, care must be taken that in the continuing arches the power of resistance be not diminished without first interposing sufficient abutment constructions to the river arches, or providing for carrying their thrust on through the land arches to their abutting terminations,—either adapting their form to carry the thrust on, or running the thrust down to their bases, as last stated, and then carrying it on by inverts. The best and most certain mode, however, is to carry the thrust of the main work at once to a firmly resisting basis, and to build whatever land arches may be required for the approaches independently of the main construction.

The flattest stone arch of large size of which the tangent of the curve at the springing is at right angles to the chord or span, is in the bridge of the Trinità at Florence, before referred to. The span of the central arch of that bridge is 95 feet 9 inches, and the rise 15 feet 1½ inch, or a little more than two-thirteenths of the span. It is built of marble, and no observable settlement has ever taken place in it, or in either of the other two arches which carry the thrust up to the abutments. The flattest brick arches of large size of which we have information are those of the bridge that carries the

struction, as the bonding of the horizontal courses in face cannot be carried through the work properly when the buttress is coursed obliquely. Joggles in the beds of horizontally coursed work will do much, and weight will have its effect, but there are objections to both.

Great Western Railway over the Thames at Maidenhead, and these have been already stated to be of semi-elliptical form of 128 feet span and 24½ feet rise, the relation of rise to span being not so far removed from that which exists in the Trinità as the difference in the density, or capabilities of resistance, of the materials employed in the two bridges respectively. The abutments of the Maidenhead Bridge are stepped on raking benches on the chalk stratum upon which they stand, and the resistance thus obtained seems to be sufficient.

Segmental arches, or arches being segments of circles less than semicircles, exercise a much severer thrust than arches of elliptical form of the same span and rise, though, indeed, the segment must be at the least 60° of a circle before a parallel can be drawn, as any semi-ellipsis within the rise of an arc of less than that proportion would present little more than corbellings from the heads of the piers to receive a merely cambered arch, and thus shorten the span a little to increase the thrust two-fold. It may be questioned whether the thrust of an arch of the form of the Trinità arches, whose rise is equivalent to the versed sine of an arc of 60° of a circle, as applied, at page 45, *ante*, to the abutments of the Chester-Dee, or Grosvenor Bridge, would exercise a more nearly horizontal thrust than the existing arch, which consists of 90° of a circle, exercises; and it is the resolution of horizontal thrust into vertical pressure that is to be sought in bridge arches.

The construction of the abutments of the Chester

Bridge is very peculiar, the vertical face or jamb of the abutment on either side being merely a prop or shore to the impost or string-course from which the arch appears to be sprung, whilst in reality the arch is carried on by radiating courses until the true span of the arch has reached 230 feet, instead of the apparent span of 200 feet, and the versed sine of the arc has become 58 instead of 42 feet. Moreover, the arch is stiffened by the extension in depth of the arch-stones in counterfort ribs through the spandrels, and these are in like manner abutted by radiating courses which carry their thrust down to and against the back piling; though, indeed, this piling is driven vertically, and does not appear therefore to be so well adapted to its duty as if it had been driven askew in a line at right angles to the bed of the stones which bear upon and press against it.

Mr. Rennie's Darlaston Bridge (Plates 108 and 109) has radiating courses in the abutments, but it will be found upon reference to the section (Plate 109) that the radiating or arch-stones in the abutments go on to complete an arch of semi-elliptical form, and spanning 102 feet, and rising 30 feet, instead of being, as it appears upon the surface (Plate 108), a segmental arch spanning 85 feet, and rising but 13 feet 6 inches. The real thrust of the arch is here carried up to the horizontally coursed wing and counterfort walls which rise from a horizontal bed of ground-piled planking, upon which the walls are liable to slip, as they are indeed within their own beds unless the courses are joggled in the beds, which is not very likely.

However soundly they may be made, and however capable of sustaining the greatest weight that can be brought to bear upon them, the foundations of bridge piers and abutments need not be loaded with the weight of useless materials. The practice, until within the last thirty or forty years, was to fill in the spandrels over the haunches of arches, and over the piers between the face spandrel walls, sometimes with earth or with dry rubbish, and at best with grouted rubble, with the intention of loading the haunches of the arches, and of forming a base for the road-way. When the sinking of the pier of Westminster Bridge took place, the injury was thought to have arisen in some degree from the weight of the rubble that had been packed in over it, and over the arches resting upon it, and the rubble was therefore removed, and contrivances were adopted for carrying over the thrust of the abutting arches by means of a hollow culvert, and a back arch springing from haunch to haunch over the pier. Perronet had recourse to something of the same kind at Orleans with the sunken pier of M. Hupeau's bridge there. A thorough perforation or cylindrical culvert was a favourite mode with Mr. Smeaton of lightening the weight on the piers, and, it would appear, of adding to the water-way also, though far too high up to be of use in that respect, whilst the arrangement was of no value in carrying the thrust of the arches from one to another. Mr. Telford introduced a great improvement in this respect, substituting coursed walls at intervals for a mass of earth,

rubble, or rubbish,[22] the walls running longitudinally of the bridge, through the spandrels, and over the piers up to the abutments, parallel to the outer face walls which carry the parapets. "These walls," says Mr. Telford, "are placed from 2 to 3 feet apart, and are made from 18 inches to 3 feet in thickness, according to their height and the nature of the materials of which they are composed,"—that is to say, diminished to about 18 inches from whatever greater thickness the height may require,—and "they are kept steady by laying long stones occasionally across from one wall to another,"—or by bonding them to one another and to the outside or faced spandrel walls by long stones at intervals when the height requires it; but Mr. Telford's plan seems to have been still to fill in the lowest angle of the

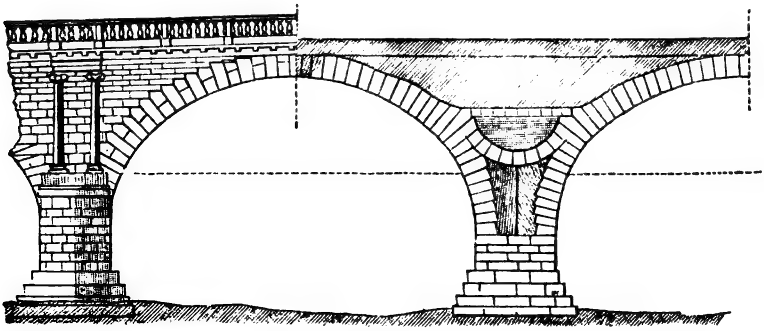
[22] In his description of Tongueland Bridge, Mr. Telford says, "In the spandrels (instead of filling them with earth) were built a number of longitudinal walls, in fact, interior spandrels, their ends abutting against the back of the arch-stones and the cross walls of each abutment: these longitudinal walls are connected and steadied by the insertion of tie-stones, and at a proper depth under the road-way the spaces between them are covered with flat stones, so as to form a platform for the road; and in these spaces are arched openings for occasional examination and repair (if ever it became necessary). I have ever since practised this mode in order to lessen the weight incumbent upon large arches, and the pressure outwards against high wing-walls and spandrels; whereas formerly they were filled with soft spongy earth or clay, in consequence of which, at the bridge originally built over the North Loch at Edinburgh (and also at other places), the side walls have been pressed outwards, and actually thrown down."—*Life of Telford*, p. 31.

spandrels and to some distance up the haunches with closely packed rubble, as shown in his disciple, Mr. Nimmo's, practice in the Wellesley Bridge at Limerick.[23] The longitudinal spandrel walls are shown in the transverse section of the Wellesley Bridge, with the bearing stone-beams or landings covering them and supporting the road-way; and such landings being of sufficient strength, or the bearing being brought within their strength by corbelling, they are better than arches turned from wall to wall, because they have no lateral thrust, as arches have, and no tendency, therefore, to force out the spandrel walls of the faces: and moreover, if the joints of the stones are effectually stopped, the backs of the bridge arches may be thereby protected from access of water by the road-way, an advantage that can hardly be bought too dearly. But, as landings to bear across under a carriage road-way may not be procurable of proper quality, arches may be turned under the carriage road, and Mr. Telford recommends the lancet pointed arch, because of its slight lateral pressure. The bonding which Mr. Telford speaks of between the longitudinal walls cannot be of much value, however, except perhaps that the bonding stones may serve as struts to steady the walls when they are of rough rubble masonry, but when brick-work or coursed masonry is used for the inner spandrel or haunch walls

[23] See Plate 55 in the longitudinal section, and in the transverse section through the spandrel of an arch, where the rubble over the pier and upon the haunches of the arches under the longitudinal spandrel walls is exemplified.

such steadying ought to be unnecessary, and the connexion between the outer face walls and the curb-stone springing walls may be better made by short cross walls as counterforts, within the length of which, being the width of the foot pavement, the thrust of the inner arches would be dissipated. It will be obvious, too, that although the solid rubble over the pier, and the longitudinal walls extending from arch to arch, as in the Wellesley Bridge, must have some effect in carrying the thrust of the bridge arches over the head of the pier and into one another, it is only by weight and friction that the material can act; unless, indeed, the backs or tails of the arch-stones were cut to present a vertical heading joint to abut the courses of the walling, when the means of conveying the thrust would certainly be something more than mere weight and friction, as in the example last alluded to, according to the representation given of it. Mr. Rennie's practice at Waterloo and London Bridges developes a different, and in some respects a better, method of effecting this, the lowermost courses of the great arches being mutually abutted or strutted over the centre of the pier by horizontally coursed gauged masonry, carried up until the flat upper curve of the semi-elliptical arches begins to quicken into the lower and quicker curves of the haunches, when an invert, springing from the tails, or back ends, of the arch-stones of the haunches, and backing on the horizontal courses over the centre of the pier, carries the thrust across in a very sound and efficient manner. Something of the same kind had been done by Mr.

Mylne at Blackfriars, but the invert appears to be used there rather to distribute the weight of the arches over the centre of the piers than to carry the thrust of the arches over for the purpose of leading it up to the abutments, though a level gauged course is introduced in the line of the chord of the invert, which has



BLACKFRIARS' BRIDGE.

that effect in some degree. This level course acts too exclusively, however, on one course of the main arch-stones to produce much valuable effect, if called upon to act at all; for it is with arches of slight rise, rather than with the almost semi-cylindrical arches of Blackfriars' Bridge, that it is so important to lead the thrust through and up to the abutments.[24]

[24] It would appear, nevertheless, from contemporary documents, that Mr. Mylne introduced the counter or inverted arch alluded to, for the purpose of carrying the thrust over the heads of the piers, and it was objected to it, with good reason, that the rubble work with which it was proposed to heart the piers, or rather the spandrels at the back of the invert down to the piers, might settle, and render the counter arches useless. In his answer to the questions proposed to several eminent mathematicians by the Committee for building Blackfriars' Bridge, in

In Waterloo and London Bridges the internal haunch or spandrel walls above the invert last spoken of are of brick-work, their duty being reduced to the support of the road-way; and as the mere weight of the road-way materials, and of the greatest traffic load that the bridges can ever be called upon to sustain, is trifling, having reference to the power such structures possess, or should possess, the fact that longitudinal spandrel walls impose the weight of the upper works upon parts only of the arches, and do not spread it over the whole surface, is evidently not of serious importance, or some evil consequence would have been developed by this time, by the sinking or otherwise of the parts of the bridge vaults in which they have been used, where the bearings are placed. It suggests, nevertheless, that bridges of the highest class may be greatly reduced in cost by their arches being turned in ribs, showing bays or coffers in the soffits, the ribs being connected by bonding stones at intervals to secure the equable settling of the whole vault, if cause be given for settlement at all. By such an arrangement one-third at the least of the stone in a vault could be spared, and a considerable further saving would result in the centering, whilst the piers would be relieved of much useless weight. The whole useful service of a bridge is to carry a road-way, and Mr. Telford's system of longitudinal spandrel walls

1760, Mr. Joseph Martyr, one of the mathematicians referred to, says—"In this counter arch I apprehend much weakness: it bearing on weak materials [rubble work] above the piers, will sink into them and be discharged of the lateral pressure it was intended to support."

shows that the road-way may be efficiently carried by less than half the constructions of a vault; for although the bonding which tends to distribute equably the pressure of the arch within itself, and to check irregular settlement, has some effect also in distributing the weight imposed upon parts of the arch over the general mass, it is truly the arch-stones upon which the spandrel walls rest that bear their weight and the weight they bring with them; the weight acts upon the arch-stones vertically when these stones are, as they always ought to be, single or thorough; vertical pressure is not conveyed collaterally; and therefore the stones lying between those receiving the weight are not acted upon in the same manner, nor to the same extent. The system here suggested is, indeed, that which prevails in timber and iron erected bridge works, and it seems almost as unreasonable to make a bridge arch one continuous mass of masonry or brick-work, as it would be thought to put ribs of timber or of iron in close contact across the whole transverse section of a bridge, instead of placing them at sufficient intervals to give strength enough to do the duty required with the requisite or expected degree of permanence. But something of the kind proposed has been practised in several instances, suggested by the necessity of the cases in which they occur, or perhaps by the recognized importance of avoiding unnecessary expense. Mr. Gibbs's bridge over the Croydon Railway, at Plate 40, is one of the instances alluded to. This bridge traverses the railway obliquely, and instead of thorough parallel abutments or springing walls, with

unsightly obliquity in every direction, and an equally unsightly askew arch resting upon them, built at great additional expense, or made infirm to avoid expense, this bridge is built in bays transversely;—what might be merely wing and counterfort becoming springing and abutment walls, to a series of ribs arranged in plan to suit the degree of obliquity. The arrangement referred to is even better adapted for bridges to carry railways than for horse-carriage road bridges, as the arched ribs may be made to fall where alone support is wanted in railway bridges,—under the lines of rails.

Bridge works of masonry or of brick-work are susceptible of another variation from the common practice, whereby the internal spandrel or haunch walls themselves may be rendered in a great degree unnecessary, whilst a great part of the thrust would be carried through a series of arches to the abutments in a more direct line than by the curved or undulating line which the inverted or counter-arch over the pier involves; the vibrations arising from heavy carriages upon the road-way would be more effectually checked than by the spandrel walls, the piers might be lightened, and, generally, great economy in the construction effected. It consists of a central longitudinal groining, which can be executed in a cheaper material than the piers, the outer faces, and the main structure of the bridge arches may consist of, being less exposed to pressure and to the weather than they are. A work of free-stone might have the more important bearing parts of a better, and the central longitudinal groining of an inferior, quality of stone, both as

to texture and colour; or the central groining might be even of rubble, or indeed of brick-work; and when the main constructions — the piers, the faces, and outer or main ribs—are of granite, the central groining might be of good free-stone, as the hearting is now made when granite is used in face. The difference suggested is perfectly justifiable as a matter of constructive propriety, since the inner groined arch would not be exposed to the same degree of pressure as the main constructions, and it would be less exposed to the corroding action of the weather, though the groin points and the returns for springing the groined longitudinal arch should be always of the stronger material.

The arrangement above suggested is slightly, and but slightly, indicated in Perronet's bridge at St. Maxence, where indeed whatever weakness could possibly arise from the introduction of a groining in the longitudinal section of a bridge, and which lies in the division of the pier into two parts, is exaggerated by the distorted form given to the divided parts of the pier, and by the introduction of small transverse arches within the springings of the main arches, so as to throw their weight, and consequently the weight of the whole superstructure, upon mere points in the piers. Plate 39 will illustrate the practice referred to, and show what is suggested.

The next question for consideration is the relation as to depth, or length in the direction of the radii, that the arch-stones of a bridge arch should bear to its span and rise; and this, like the former questions as to constructive composition, must here be considered empirically.

In the whole direct sweep of an arch there is no one stone of greater or of less importance to the stability of the structure than any other: if a springer or a haunch-stone should fail for want of sufficient extent of bed, or for want of sufficient denseness of texture in the material, the arch must as assuredly fail as if the key-stones were crushed by the pressure upon them of the two halves of the arch. If it be considered that the aim in arranging and constructing a stone bridge is to produce the nearest approach that can be made, by shaping, combining and connecting the main constituent material, to a figure of the best adapted form hewn out of a solid mass of the same substance, and reduced to the minimum of quantity, —the form and quantity requisite to the desired effect being ascertained and determined by mathematical investigation,—it will readily appear how important it is that every care should be taken to combine the parts in such manner as to produce immobility. Mathematical resolutions are valuable in the composition of bridges in proportion to the excellence of the work to be applied in their construction, or rather, perhaps, to the unyieldingness that can be produced in the construction, whatever may be the process; but it is quite certain that any movement of the parts, or in the parts, not premised, or not exactly as premised, must destroy all confidence in arrangements as to form, quantity, or position, established on mathematical bases. The crowns of the arches of Perronet's Neuilly Bridge sank 23 inches, so that if the curve of equilibration due to the structure fell within the substance of the arches as constructed,

it must be now very much out of its place with regard to the great bulk of the material used in the construction of the arches respectively; and whatever value was attachable to the particular curves laid down for the arches is necessarily lost, or transferred to the hap-hazard curves into which the stones settled down. But there was no necessity for the effect that happened: the piers did not sink with the weight of the arches, for they had borne them with the weight of the centering added to that of the arches, and the abutments could not have yielded, for if they had moved at all, nothing existed to prevent them from going far enough to let the arches down altogether:—it was simply bad masons' work in the beds of the arch-stones, which admitted, or rather required, mortar, and other compressible packing, in the joints, to supply the place of solid stone that had been cut away from the beds of the arch-stones.

The arches of London Bridge are flatter in the relation of rise to span than those of the bridge of Neuilly, and are of greater extent, but they settled down at the crown not more than a tithe of the extent stated of the latter. There is no other reason why this should have been so, but that referred to;—the difference between good and bad masons' work. But, in neither of these cases would settlement have taken place if sufficient time had been allowed before the striking of the centres to allow the mortar to indurate to the degree requisite to enable it to withstand the pressure without yielding, as good mortar will acquire strength enough to bear the weight

that had to be resisted in either of the instances referred to, though the time would be long in proportion to the thickness of the mass and the degree of induration required to withstand the pressure in each particular case, and though the mortar might never acquire one-tenth the degree of hardness of the stone set in it.[25]

The objections stated in the Preliminary Essay to this Treatise to brick-work and rubble masonry in bridge constructions, because of the large proportion of mortar required to both, have reference to the strength of the main constituent in each class of construction, brick and stone, and the remarks introduced are intended to show that reliance is not to be placed upon such work beyond what may be due to the power of the mortar to resist pressure. Mortar and concrete, when properly composed and well compounded, acquire in time many of the qualities of ordinary free-stones, and a bridge might be formed or cast in a connected homogeneous mass of either substance that with sufficient time would be better and more enduring than a constructed work of stone of like quality; but within the time required for induration such moulded formation must remain cased in its moulds and supported by artifice. To shorten the period of induration and to render casing unnecessary, by depriving the mass of fluidity, the mortar is reduced

[25] Perronet says that the operation of striking the centres of the bridge at Neuilly, by the removal of the immediate support of the arches, was commenced eighteen days after the key-stones had been put in their places!—Mémoire sur le Cintrement et le Décintrement des Ponts, &c.

to the smallest possible quantity, by the admixture with it of already indurated substances, as bricks, or as stone in the form of rubble; and that the best effect of which they are capable may be derived from the presence of these substances, they are artificially arranged by bonding, and so as to reduce the mortar into as nearly parallel plates as the forms in which the substances present themselves will permit; preserving, nevertheless, the perfect connexion of the mortar with itself within the substance of the work, since, in this view of the case, the mortar is the basis and main constituent of construction, upon the power of resistance of which the retention of the form given to the composition in which it is employed entirely depends. Let such a composition be supported by unyielding centering until the mortar shall have indurated sufficiently to hold its own when exposed to the remaining load of the upper works and road-way, and to the weight and vibratory action of the intended traffic, in addition to that inherent in itself, and not the slightest settlement of the so-formed arch will take place; and such an arch, moreover, will be more perfect and more immediately trustworthy than one built of strong stone imperfectly wrought in the beds, or wrought with open joints and committed to the packing and bedding mortar while this is in a comparatively green state, as the latter would settle and the former would not.[26]

[26] While writing these pages, the attention of the author has been called to a practical instance confirmatory of these views with regard to the mischievous effect that may arise in constructions from the

When the circumstances of the case are such that rubble masonry depending, as above stated, upon the mortar, is strong enough to carry and resist all that is required,—and for ordinary road bridges this will often be the case,—the work may with advantage be protected on each face from the wind and rain, by quoin and archivolt ribs of wrought and gauged masonry. These should bond themselves by headers into the rubble work of the soffit of the arch, and being bonded together across the whole breadth of the soffit at intervals by parallel gauged courses, of which the stones may with advantage be cramped together to effect a transverse tie, the rubble work would be framed, and resolved

yieldingness of unindurated mortar. Upon an investigation of the state and condition of the noble church of St. Mary Redcliffe at Bristol, the beautiful attached tower was found to be curved outwards upon all its faces beyond the line of the vertex, to the extent of 6 inches in 50 feet; that is to say, the walls of the tower, instead of being upright, are bowed outwards in an arc whose versed sine is 6 inches, the length of the chord or true face of the wall being 100 feet; whilst, at the same time, the walls and buttresses of the church, where there is no settlement of the ground under them, remain perfectly upright. Now the walls of the tower consist of close-jointed gauged masonry on the outer faces, hearted and backed up to the inside with coursed rubble, whilst the walls of the church are substantially of gauged masonry throughout, both faces being wrought; and bonding stones occur so constantly as to connect them perfectly, and take off all dependence upon any rubble hearting. Hence it would appear that the rubble hearting and backing of the walls of the tower yielded to the weight of the superstructure as it accumulated, whilst the wrought masonry of the outer faces, being incompressible, was forced to assume the longer curved line, to adapt itself to the rubble; and as the backing is of the same sort of stone as the ashlar facing, it was the larger mass of yet unindurated mortar that became compressed, and occasioned the imperfection that exists.

by the framing into coffered panels. Arches may, however, be built of wrought and gauged masonry, disposed in ribs at intervals transversely, to make up the breadth of a bridge, as before suggested, instead of continuous vaults, so cheaply,—especially where strong landings to cover them can be procured at an easy rate,—as to make it not worth while, for works of any pretence, to use the inferior composition with the greater weight that its continuous mass imposes upon the piers, considering its greater liability to degradation and decay from the agencies to which bridge works are exposed.

A large proportion of the substance required in the crowns of bridge arches might be dispensed with if such works were not liable to be acted upon violently by heavy bodies in motion, and more particularly in rapid motion. It is desirable, therefore, to make such arrangements as may tend to reduce the effect of this vibratory action to the lowest possible degree, and to distribute what may remain over the greatest possible extent of surface, that its effect may be reduced to harmlessness, if it be not wholly dissipated. How far the nature of the case will allow this to be done must be considered in determining the substance to be given to the structure of the crowns of the arches of a bridge, and particularly the depth, or length vertically, to be given to the keying courses; for the power of resistance to the pressure induced by the mere weight of the structure itself, and of the load that can be brought upon a bridge, of any stone that will withstand the action of water and the other corroding influences to which bridge works are for the

most part of necessity exposed, is so great as to allow of the structure being made exceedingly slight, if the stones are brought together with truth, and adapted in their form to the form of the arch, every stone bearing in itself the proportion between the inner and outer peripheries of the substance of the arch where it occurs. It must be considered, too, what portion of the arch will press upon the key-stone course, and in what degree, taking it as a general rule that stones wrought with that degree of smoothness which should be given to the beds of arch-stones, begin to slip upon one another when the bed or inclined face is at an angle of 30° with the horizon, the beds being free from either lubricating or adhesive substance to facilitate motion or to restrain it. With the tendency to slip commences the pressure of the arch-stones upwards; but as no course in an arch, not even that next the key, can press with the whole of its gravitating force upwards, the pressure communicated to the key-stone course can only be a part, and a calculable part, of the weight of those portions of the arch which press upwards at all; such portions being limited in a semicircular arch to two-thirds of the whole quantity as a maximum. Moreover, the adhesiveness induced by the presence of mortar in the beds of the stones will carry the angle of friction much higher up than the angle above indicated, and the bonding of horizontal courses in the spandrels tends to further this effect, whilst these, combined with joggles in the beds, may be made so greatly to reduce the pressure towards and upon the

key-stone course, and within the crown or the upper and flattest parts of an arch, as to ease the centering in a very great degree of the effect of a load in the haunches, and free the key-stone course at the same time from so much undue pressure. Such considerations may have their weight when circumstances dictate the use of stone of inferior strength and in blocks of small sizes; but with a strong material, and a sufficiency of means to apply labour enough to insure perfect work in the beds, it will be enough to derive from them greater confidence in reducing the thickness of the crown of an arch to the lowest degree consistent with perfect security in every respect, and with a due regard to the durability of the work.

As the position in an arch of the key-stone course is such as to require that it should not be deeper than the various circumstances connected with the material, the structure, and the duties imposed upon, and the services required of it, demand, the depth or length of the key determines the substance, or the limit of thickness, of the crown of an arch; since all the other courses in the crown—supposing the road-way not to fall more rapidly at least than the arch rises—may be certainly as deep as the key-stone course without any inconvenience, and for the most part they may be increasingly deeper, if it be deemed necessary to make them so. But the circumstances are so various that no general rule can be established, and every individual case must be settled with regard to its particular circumstances, having reference to the various considerations hereinbefore

suggested. The span of the arch is necessarily an important point, but the rise, and the form in which the rise takes place in reference to the span, the strength or power of resisting pressure of the stone or other material to be employed, the quality of the work to be applied in dressing, or otherwise forming, the beds and bringing the parts together, the stress and the vibrations to which the completed structure may be liable, and the means to be used and applied in distributing weight and dissipating shocks or other tendencies to disturbance, must be all considered in relation to the span and to one another. The bridge over the Serpentine water in Hyde Park and Kensington Gardens may be loaded with human beings, and horses and other cattle may trot or even canter and gallop over it, but it is protected from the action of wheeled carriages heavier or faster than the road-makers' or scavengers' carts at long intervals;—the Maidenhead Railway Bridge is free from liability to be heavily loaded by concourses of men or by herds of cattle, but is exposed to the powerful action of the twenty-ton locomotive engine with its enormous train in rapid and agitating motion;—and London Bridge, with its mixed and generally most exciting traffic;—these admit of, and indeed require, very different modes of treatment.

The heaviest load in dead weight that a bridge can be exposed to is that arising from a dense crowd of men, and the most exciting action that a body of men can induce in using a bridge arises from the march of troops; but this could not be felt by a constructed work of brick or stone, because of the distribution of

the action, and the consequent diffusion of the effect, and of the absence of concussion attending that action that could be felt in a mass of dense and practically inelastic materials. The weight of the materials pressing upon the parts of the arches, whether within themselves or in the upper works and road-way, and the weight of as many adult human beings as could stand crowded together upon that part of the road-way over any particular arch, will give the greatest stress the work is likely to be subjected to in the case of a bridge like that over the Serpentine; but it would be unwise to cover that effect merely, and three times the substance requisite to do so will not be too much to embrace all contingencies. The case of a railway bridge admits of easy calculation as to the heaviest weights to which an arch may be liable, and the concussive action and vibratory effect of the load should be met and counteracted, at the same time that the weight is distributed, by means for diffusing it, and so destroying its exciting effect. Longitudinal spandrel or haunch walls tend to do both, and if these are coped with long stones as trams, and the rails bear upon the trams with longitudinal sleepers of timber intervening, the timber being of large scantling to distribute the weight as a wall plate does, the shock of the concussive action of the engines will be diffused over the whole substructure. London Bridge cannot be loaded more heavily than by men in a dense crowd; but, as with a railway train, particular points upon it may be loaded greatly beyond what can be imposed upon the average extent of its surface; and in this case

also,—superabundant strength being provided to carry the weights that may be brought to bear upon the constituent materials,—means should be adopted to distribute the weight and to diffuse the effect of concussions arising from heavily laden waggons and fast and heavy carriages, as stage coaches and omnibuses. The whole should be floored, in a direction parallel to the intended surface of the road-way, by landings bearing upon the longitudinal haunch walls, and clear of the structure of the arch in every case, to distribute the weight equably, whilst a stratum of inelastic material, as chalk-rubble or other similar substances not liable to crack when dry nor to run into mud when wet, and covered by dry brick rubbish, hard core of potsherds, or strong gravel, should form the bottoming for the road material, whatever that may be. A binding material, as broken stone or clean gravel, is the best, and pitching stones the worst, to prevent vibrations from being carried down into the structure.

In settling the important point now under consideration, the serious error of reckoning upon the strength or power of resistance of a stronger material, when the duty is really to be imposed upon a weaker, must be carefully avoided. If the arches of the free-stone bridge in Hyde Park bear upon the mortar in their composition, and not upon the stone, the substance given to the arches may not be unnecessarily great; but if the stone be brought to bear in effect, by the mortar in the beds being reduced to the smallest possible quantity and in parallel layers, the substance of the vaults is, for

the duty required of them, probably uselessly large. In such a composition as the Maidenhead Railway Bridge, where the dependence is entirely upon the substance used as mortar, the bricks merely serving to give consistence to the otherwise friable mass, by reducing the mortar to layers, but layers of such substance and disposition, nevertheless, as to leave the structure entirely dependent upon its power of resisting the pressure and vibratory action of the whole work and of the load, the power of the weaker constituent of the composition must be considered the datum in computing the substance in depth of the arches. In the case of London Bridge the granite itself may fairly be reckoned the main constituent; and the arches of that work will be found, upon calculation of the strength or power of resistance to pressure of the material, and of the stress to which it is subjected, to be greater in substance at their crowns than the exigency of the case required, all contingencies being fully allowed for.[27]

[27] Mr. Thomas Simpson, of Woolwich Academy, another of the "eminent mathematicians" referred to by the Committee for building Blackfriars' Bridge, says of Mr. Mylne's design: "As to the upper part of the arch, from the key-stone to the haunches, the strength thereof is exceeding great,"—as he had before signified,—"when the piers and every thing below the haunches is considered as immoveable. But this excess of strength, arising from the length and disposition of the voussoirs [arch-stones], appears to be attended with a real disadvantage when we come to consider how much the lateral pressure is by this means increased. Were the length of the arch-stones at the key to be only 5 feet or 4½ feet instead of 6, and at the haunches 7 feet instead of 8, the arch itself would even then have greater strength than will be necessary to support much greater weight than ever can be brought

For the reasons already stated, tables of the leading dimensions of arches executed in bridges convey but little useful information in the absence of such critical particulars in every case as it is found impossible to supply with sufficient authority to make them of value. Such tables would state that the arches of the Maidenhead Railway Bridge and the arches of the bridge of the Trinità are of elliptical form, and that the former work is of brick and the latter of marble.[28] It would appear, from stated dimensions, that the thickness of the Maidenhead arches at the crown is rather less than one-twenty-fourth the span, and rather more than one-fifth the rise; and, indeed, that this thickness, which in a stone arch would be called the length or depth of the key, is the nearest approach the brick-work will allow of to the

over it. . . . . . By this means also the lateral pressure would be diminished near one-quarter part, and consequently a less breadth of piers would be necessary, which would not only considerably lessen the expense, but likewise be of advantage to the navigation." Probably this opinion was the means of reducing the key-stones from the intended depth of 6 feet to 5 feet, but in the late operations upon Blackfriars' Bridge, for the purpose of lowering the road at the summit, the key-stones of the central arch appear to have been reduced below even the smaller dimension suggested by Mr. Simpson.

[28] The facing courses of the Trinità arches are of wrought and gauged marble, and transverse courses of wrought stone of the same sort bond the faces at intervals, but the spaces between the bonding courses are said to be filled in with rubble masonry; if this be so, however, the rubble must have indurated sufficiently to carry the pressure of the arch within itself before the centering was removed, as ought to be the case, since the rubble has settled no more than the gauged masonry itself, which is so trifling as to be almost, if not altogether, inappreciable. See *ante*, pp. 156, 157.

span divided by the rise ($128 \div 24{\cdot}25 = 5{\cdot}278$), the real dimension being 5·25 feet. The dimensions would show the thickness of the crown of the central arch of the Trinità to be one-thirty-second the span and one-fifth the rise, differing from the Maidenhead arches in the one as the difference in the material might appear to justify, but agreeing with them in the other proportion; whilst a division of the span by the rise in this case would give 6·33 feet for the depth of the key, whilst its real dimension is but 3 feet, showing a difference which nothing appears fully to account for. Moreover, the relative span and rise of arches of various forms,—semi-circular, semi-elliptical, and segmental,—give a very deceptive view of the comparative pressure of arches within themselves.

It may be useful, nevertheless, to show that the men who are esteemed among the ablest practitioners had no settled rule in their own practice,—or that, if they had, it does not appear from the works executed by them,—in establishing the relation between the dimensions of arches and the depth of their key-stones respectively, even when the nominal constituent material was the same, the form of the arch or arches the same in effect, and when the duty to be performed, or the weight and character of the load to be borne, were not dissimilar; as a consciousness of this may lead to a conviction of the necessity of placing every case upon its own merits. Perronet's bridges at Neuilly, at St. Maxence, and at Paris (Louis the Sixteenth's), and the bridge designed for Melun, were all built, or intended by him to be, of

stone from the same quarry,—that of Saillancourt, near Meulan on the Seine,—the arches are all in the form of small segments of large circles, except the first, whose faces are, however, of that configuration—and only that part of the inner ellipsis which is above the chord of the segments of the faces can be included in the comparison,—and they are all in the lines of great public roads, and in or near populous towns, and therefore exposed to the same actions. Taking the depth of the key-stone course, or thickness of the crown of the arch in each case, as unity, the relation that depth bears to the span and rise of the arch may be stated as follows:

	Depth of key.	Rise or versed sine of arc.	Span, or chord of the arc.	
Neuilly (of five equal arches, 127 ft. 10½ in. span)	1	2·9	24 22·5	Outer segmental arch of faces. Corresponding part of inner arch.
Paris (middle arch of five arches 93 ft. 9½ in. span)	1	2·8	25·66	
St. Maxence (three equal arches 76 ft. 9 in. span)	1	1·765	21·53	
Melun (one arch 159 ft. 10½ in. span)	1	2·272 2·9	32 29·54	Outer arch of faces. Inner arch of crown.

The only stone bridges built by Telford that admit of comparison in the manner proposed are Tongueland Bridge and Gloucester Over Bridge. These are both built of sandstone, of similar qualities, and are both upon main lines of carriage road, open to any service to which such lines are exposed. The arch of the former (both are of one arch) is in the form of a segment,—not a small one, certainly, if compared with Perronet's flat segmental arches,—of a not very large circle, the span

being 112 feet and the rise 35 feet; the arch of the latter is of the complex form of the bridge at Neuilly, the crown semi-elliptical, and the faces in the form of a segment of a circle coincident with the flattest part of the ellipsis; and so much of the crown of the elliptical arch is taken here as lies within the chord of the segmental part of the arch.

	Depth of key.	Rise or versed sine of arc.	Span or chord of arc.	
Tongueland (112 ft. span)	1	10	32	
Gloucester Over (150 ft. span)	1	2·89	33·34 26	Outer segmental arch of faces. Corresponding part of inner arch.

To bring Rennie's great works over the Thames at London into comparison with one another in the required point of view, it is necessary to take those portions only of the upper parts of the semi-elliptical arches of Waterloo and London Bridges that fall within 60° of the circles they respectively approximate, as this excludes those parts of the arches at the springings, and up into the haunches, of which the pressure upwards is slight, though it certainly includes more than is included of the Perronet and Telford complex arches at Neuilly and Gloucester. But the object here is to compare the practitioner with himself and not with others; and, indeed, the difference in density of the primitive rock in the two London bridges, and of the sandstones used at Neuilly and Gloucester, prevents any comparison between the arches of the one and of the other:

	Depth of key.	Rise of crown in arc of 60 degrees.	Span, or chord of arc of 60 degrees.
Waterloo (nine equal arches of 120 ft. span)	1	2·63	20·63
London (middle arch of five arches 151 ft. 9 in. span)	1	3·37	26·95

It is not advisable to increase the length or depth of the arch-stones which press upon the keys in any great degree, as any increase in their size necessarily increases their gravitating force, and gives them thereby a tendency to fall through, which the shorter stones at the summit of the arch may not have weight enough to check. It is desirable, moreover, to introduce the horizontal coursing over the haunches of an arch as high up as it can be done, consistently with the retention of sufficient length in the intercepted arch-stones, for the purpose of distributing the weight of the upper works and load over the arch most equably, as well as for the purpose, before pointed out, of aiding to carry the thrust of the arches from one to another, and up to the abutments. Where the crown of an arch is flat, it seems necessary to allow the collateral arch-stones on either side of the keys to increase in length, until the arc shall have increased sufficiently to allow of the introduction of an horizontal course over the tail of an arch-stone without making it less in depth or length upon the radius than the depth of the key-stone course at least. This is exemplified by exaggeration in the elevation of Staines Bridge (Plate 53), and in that of the bridge of the Champ de Mars (Bridge of Jena, Plate 56). In the former work the horizontal

coursing seems to run in over the arch-stones too far, giving an appearance of weakness to those which are intercepted; and in the latter the converse effect is produced: the collateral arch-stones of the crown become heavy as they advance upon the haunches before the horizontal coursing runs in over them. Sir Robert Smirke has endeavoured to meet the objections which arise in this respect from the slowly retiring soffit of a flat-crowned arch in the Westgate Bridge at Gloucester, by halving the upper horizontal courses, to stop the tailing upwards of the arch-stones of the crown and haunches earlier, and so to give the structure of the arch a more regular form than it obtains otherwise. This practice is exemplified in an arch of the bridge over the Eden at Carlisle, (Plate 52,) but it does not obviate the difficulty, except by imposing a more disagreeable effect to the eye; whilst the distribution of weight and vibration, and the carrying on of the thrust of the arch-stones, are not so well intrusted to the thin shelf-like ends of the horizontal courses as to the bolder coursing and abutting exhibited in Staines Bridge. Darlaston Bridge (Plate 108) shows a better arrangement in this respect, though that is susceptible of improvement, independently of the very objectionable mode of springing the arch adopted here, and so well avoided in Staines Bridge. In Ballater Bridge at Aberdeen (Plate 1), and in the Hutcheson Bridge at Glasgow (Plates 27, 28), the constructive advantage derivable from abutting the arch by the horizontal coursing in detail is altogether thrown away, and the masonry of the spandrels acts by its weight

alone; but this arrangement is not so unsightly nor so faulty as that exhibited in the faces of the bridge across the Forth at Stirling (Plate 62). Westminster Bridge (Plate 24) exhibits the arrangement last referred to as faulty, and first spoken of as not advisable. The means of distributing weight and vibration over the haunches of the arches by horizontal layers, as courses, are altogether thrown away, and the arch-stones are tailed out to an extent that would in bolder arches be mischievous, though in the present case the construction is fortunately only ridiculous. In Blackfriars' Bridge the system of abutting courses is introduced with good constructive effect, though the arches are injudiciously and most unnecessarily built, or they appear to be built, in header and stretcher courses, and the halved horizontal heading course used to so great an extent in the Westgate Bridge at Gloucester and in the bridge over the Eden at Carlisle, is here slightly developed. An example of really good arrangement of the tailing of the arch-stones with the horizontal coursing of the spandrels, in a case difficult of management, because of the slightness of the rise of the arch, is found in the bridge over the Dora near Turin, and both Waterloo and London Bridges are very well arranged in this respect as to their faces. The arrangements referred to, whether for good or evil, are, however, generally upon the surface only; but these remarks are intended to apply to such arrangements as if they were carried into the construction, as they are shown upon the faces.

An efficient system of abutting the tails of arch-stones

as they rise in the haunches of an arch by horizontal bonded courses, which may also distribute the weight of the load and the vibrations arising from its concussive action over the structure of the arch, aided by accurate work in the beds and joints of the stones throughout, would be most requisite in building arches in ribbed series, instead of continuous vaults, as suggested in an earlier page of this treatise. The invert used by Mr. Rennie, and before referred to, is excellent as a means of carrying over the thrust of the flat crowns of arches, but the mode of construction which it involves does not distribute the weight and vibration of the load, as the longitudinal abutting courses do. Moreover, it is hardly available with brick arches without dropping the backs of the courses so as to give the effect of weakening the haunches in some degree; and when the work is in rings, and not bonded through the thickness or depth of the arched construction, such invert might have the effect of separating the parts of the arch,—of ripping the back off and leaving the inner part of the soffit to settle down. Nor does brick-work, for this last reason, give the means of bringing the horizontal coursing into operation with the same effect that masonry does, and the spandrel and haunch walls can only act upon the arches to carry the thrust over, as they do in masonry construction when the tails of the stones do not run up to abut upon the heading joints of the horizontal coursing of the spandrels. The equable distribution of weight, and the deadening of concussive effect from the load, are equally obtained with

brick-work as with masonry, from bonded horizontal coursing.

The arrangement of the essential constructions of a bridge of masonry being thus considered with reference to their capacity and form, in relation to the desiderata in bridge building—the protection of the water-way for its useful and necessary services, and the obtaining a secure and convenient road-way over it at the least expense—it remains to consider the form that should be given to the ends of a bridge pier in a water-way, to the end that the least disturbing effect may be produced upon the current by the presence of the pier, both for the protection of the pier itself with the constructions resting upon it, and for the safety of the navigation, when the water-way is navigable. As far as the support of the constructions is concerned, the pier need not be extended in the direction of its length, or in the direction of the transverse section of the bridge, beyond the faces of the bridge itself; but as a pier in a water-way is liable to be acted upon in a greater or less degree by the water in which it is placed running in a stream, in a torrent, or by the operation of the tidal influence, that form must be given to the ends of the pier which shall have the effect of parting the water, and of allowing it to re-unite in the least disturbing manner. The regular parallelogramic form which a bridge pier bears in its plan would present a straight line, in length equal to the thickness of the pier, and at right angles to its sides and to the current; and as no

alteration can be made in this form within the plan of the piers, the required cutwaters[29] must be obtained by adding to the length of the pier whatever may be necessary to the effect proposed. The question then remains, what form of extension is best adapted to the service? — the amount of the extension will depend almost of necessity upon its form. The bows of a ship may be thought to dictate the best form for the cutwaters of bridge piers; but as ships adapted for sailing are generally understood when ships are spoken of, it is not true as it regards their bows or cutwaters, since a bluff form is necessary to enable a ship to sail upon a wind, or to work to windward,—a quality most essential to all sailing vessels; but the best form of the bows for a steam-vessel or row-boat is that best adapted for the cutwaters of bridge piers; and, indeed, the general form of such vessel is the best for the particular

[29] The term cutwater applies very well to the projections upon the ends of piers in tidal streams, as both ends are in turn opposed to the current and act as *cut*waters; but where there is only the down-stream current, it is the up-stream end of the pier alone that can present a *cut*water, though truly the same, or nearly the same, form is required at the down-stream end to bring the divided water together without forming dangerous eddies. The French, most of whose bridges feel the stream but one way, term the up-stream side of a bridge the *côte d'amont*, or the side upon which the water mounts, and the down-stream side the *côté d'aval*, or that on which the water in meeting forms a valley or depression. The French have, too, distinctive terms for the extensions of the piers, to which we, with our tidal rivers, apply indifferently the term cutwater. The true cutwater is called the *avant-bec*, the beak or prow before, and the down-stream projection the *arrière-bec*, the beak or prow behind.

purpose of allowing the water to go by it with the least disturbance. The more important duties of the pier require, however, that it should be of a different form in the body, though its extensions may take the form of the bows of a whale-boat, but without running away to a keel, as the cutwater has to divide the water throughout its whole depth, and not to the depth alone that a boat may draw.[30]

Besides dividing the water, the cutwaters of bridge piers have generally another important duty to perform, which requires that they shall have strength to resist concussions, whilst their form may have the effect of turning off solid substances, as well as of dividing the fluid in which they stand. Most rivers are liable to be frozen over at times, or to be otherwise encumbered with masses of ice, which thick and square-ended bridge piers will detain as they float down upon the current; a block of ice once arrested will rather detain others than be itself set in motion again; so that, in the absence of proper cutwaters to the piers, a dam of ice may be formed across the river through the agency of the bridge. Rafts of timber and uprooted trees also, and craft navigating a river, should find cutwaters in advance of the piers of a bridge, of strength sufficient to receive any blow they may communicate without sustaining injury from the blow, and of a form adapted to throw off the obstruction.

[30] This subject may be studied with advantage wherever vessels, and particularly vessels of considerable draught, lie at anchor by the bows in a current.

The form best adapted to divide the water in its current is also well adapted to throw off obstructions, but as the cutwater must possess power also to resist blows, the best adapted form must be restricted in its proportions to obtain for it the requisite degree of strength. The bows of a whale-boat are, in horizontal section, in the form of an acute angled isosceles triangle, with the legs of the triangle slightly curved convexly; and this form, given in extension to the ends of a bridge, in such manner that the end of the pier shall be the base of the triangle, and the tangents of the curved sides of the triangle at right angles to the base, will be more or less efficient as a cutwater, as the angle at the apex is more or less acute. A great degree of acuteness would lead to extravagant elongation of the pier, and to weakness in the cutwater; so that the circumstances of each particular case must be taken into consideration to determine the degree of acuteness that may be desirable, and the amount of strength required, with reference to the cost of combining them; and the advantage to be derived from incurring the expense of elongating the pier to produce acuteness in the cutwater, and of giving to the cutwater strength beyond that which the stone used may possess.

Almost every variety of shape of which they are capable has been given to the cutwaters of bridge piers, and cutwaters have been more frequently formed according to the designer's fancy of what was handsome than with reference to the useful service required of them. All observation and experience show, however, that the

form above developed is the best adapted to all the duties of the cutwater; and the particular proportion most commonly preferable is that of an equilateral triangle, inscribed in curves struck from the two ends of that side which is upon the end of the pier, the radius of the curves being of course the length of that side. This proportion fulfils all the conditions required, dividing the water without greatly disturbing it, and giving strength to the stone upon the salient angle.

As the arches of a bridge should never be sprung below high-water level, so should the cutwaters never be raised above the level of the springing of the arches, that the projecting ends of the piers forming the cutwater may be bonded into the structure of the piers themselves. According to the vicious practice of springing arches within the range of tides or of ordinary floods, the cutwaters must be carried up before the faces of the work, without being bonded into it, or the bonding must be effected by the unskilful and expensive mode of cutting down upon the blocks of which the arch-stones are formed to raise tailing ends that may run out upon the cutwater in horizontal courses. Mr. Rennie has left an excellent example in Staines Bridge of the best mode of combining the cutwater with the pier, by making the semi-domed head of the cutwater run in under the springing courses of the arches; whilst London and Waterloo Bridges exhibit in the same particulars a mode of practice carefully to be avoided.

Although the width of a bridge is comparatively un-

important, as a matter of construction, provided it be sufficiently wide to give its piers, with the added cut-waters, length enough to withstand any action to which the work may be exposed from floods or otherwise, it may be proper in a practical treatise upon bridge building to give a few hints on this head.

Carriage roads should always be wide enough for three carriages at the least to travel upon the same transverse line at the same time, that fast carriages may not be unnecessarily impeded by slow ones, and that the road may be open even if two carriages happen to be stopped on opposite sides of the road at the same time. In carriage roads, as streets within towns, where there may be dwelling-houses, shops, or warehouses, on both sides, before which carriages must be frequently standing, and on both sides at the same time, there should, for the reason before mentioned, be width between the standing carriages for three at the least to move abreast, and where the thoroughfare is great, even more is greatly desirable. Light carriages cannot be required to stand in less than 6 feet, nor can they be driven with freedom and safety in less than 8 feet; heavy carts and waggons should have 8 feet to stand in, and 10 feet when in motion; so that no public road of mixed traffic ought to be less than 28 feet wide in the carriage-way, and no street that is a carriage thoroughfare, and built, or liable to be built, on both sides, should be less than 42 feet wide in the carriage-way: much used thoroughfares, as all streets leading to and from a free bridge within a town must be, because of the concentration

of traffic from many streets to that one street, should of course be still wider. According to the circumstances of the approaches, then, rather than according to their actual width, should be the road-way upon a bridge. A public road bridge in a line of road not liable to be greatly obstructed by carriages standing by the sides, should be as wide as the road ought to be, that is, wide enough for three carriages, taking an average of fast and slow, to pass abreast; and when a bridge stands within a town, the space occupied in the streets or other approaches leading to it, by carriages standing by the sides, may be deducted from the width the street ought to have, for the carriage road-way of the bridge. The foot-ways on a bridge should, however, be as wide as they are required in the streets or roads leading to or from it; as the inducement to stop, and thereby to obstruct the way, is at least as great on a bridge as in a street or road.

It has been already stated that the road-way should be borne over the main constructions in such a manner as to distribute the weight and the vibrations arising from the load and from the working, in an equable manner over all. The old mode of filling in the spandrels with earth or rubbish in a mass was connected with a further filling over the backs of the arches also, to interpose a body of matter through which the vibrations arising from the working of the road upon it did not readily pass; and in abandoning the old mode provision should be made for retaining the advantages it possessed, of which this was not an

unimportant one. If the effect of the jarring of heavy and fast traffic upon a bridge be communicated to the constructions they must suffer from it. Although the stone itself be practically inelastic, the mortar in which it is set is seldom so; a flaw in a stone or other material may be acted upon until a piece is thrown off,—mortar becomes loosened, then detached,—and such is the progress of works to destruction, even when originally the most rigid in their construction.[31]

[31] An illustration of this is found in the celebrated stone arch between the eastern walls of the west front of Lincoln Cathedral, and known as the Lincoln beam, because of its flatness, or slight rise in proportion to its span. In a Report on the construction of this arch by Mr. W. A. Nicholson, Architect, of Lincoln, printed in the Transactions of the Royal Institute of British Architects, (Vol. I. Part II. 4to., London, 1842,) the author says: "The arch vibrates perceptibly when jumped upon; and I am of opinion that the constant practice of visitors thus to prove its elastic properties has a tendency to impair its stability. The mortar between the joints is of a very friable nature [all kinds of mortar are so in a greater or less degree], and has detached itself from the parts where it has been used in pointing the joints to conceal the irregularity of the workmanship; and it may be still further crushed by the violent concussions to which the arch is almost daily exposed." It may be remarked that this "beam" is not so flat as equal portions of the crowns of most of the larger semi-elliptical bridge arches. The Lincoln beam spans 28 feet, and rises 14¼ inches, whilst the crown of the central arch of the Trinità rises barely 12 inches upon a chord 28 feet long. It is true that the substance of the arch in the bridge is more than three times that of the beam; but 28 feet is less than 95·79 feet, the span of the whole arch of the bridge, in the same proportion, within a fraction, that the substance of the arch of the one is less than that of the other; and whilst the beam is exposed to nothing more violent than a man jumping, the bridge is constantly liable to the rolling upon a stone paving of heavy waggons, and of fast carriages

Horizontally coursed work should, therefore, be introduced over the backs of arches in longitudinal walls, some inches high at the least, in the shallowest part, to bear off the bridging stones or landings from the substance of the arches, and to distribute the weight, so that concussive action may be distributed also, even if it be not entirely dissipated. The same effect should be still further pursued by the introduction over the floor of landings of an uncombined substance, as hard core, under any stone paving; though well executed wood paving, consisting of blocks bedded on an oblique, or on a direct transverse, section of the fibre, especially if dowelled together, might be laid at once upon the landings.[32]

drawn by trotting horses. But the crown of the central arch of London Bridge does not rise so much upon a chord of 28 feet as the Trinità arch rises; and the Maidenhead Railway arches appear to rise even less than the London Bridge arches!

[32] Having mentioned wood paving, it may not be out of place here to intimate that the practice commonly pursued with the obliquely cut wood paving lately introduced into use in London, for the purpose of obviating the objection to its presumed slipperiness, is the farthest removed from the true practice with such a material, and quite destroys the only advantage derivable from cutting the blocks in an oblique section at all. This advantage is the brush-like surface the obliquely cut fibrous mass presents upon being slightly disturbed by the working out, on pressure, of the hardened albuminous matter that combines it. Such brush-like surface has the roughness necessary to prevent horses' feet from slipping upon it; but being clogged with dirt and wetted, a greasy, and consequently slippery, surface is produced. It should, therefore, be kept clean, in which condition it is perfectly safe; but while the stupid practice prevails of distributing sand over the paving, it can never be free from dirt, and will, consequently, be unsafe for

All roads should be so composed and arranged as to be as nearly as possible impervious to water, or to be relieved of water that may pass through the surface by efficient under drainage, for the sake of the roads themselves; but the road-way upon a bridge should be so composed and arranged as to carry off all that may come upon it by surface drainage alone, when the nature of the case renders it possible to do so, for the sake of the works of the bridge, as well as for that of the road upon it. As this in ordinary practice cannot be done completely, means of collecting and discharging water that may percolate the road should be provided within the spandrels and over the piers. Puddled clay has been often used for covering the work, but as this substance is liable to become dried, in such situations, it cannot be trusted, and bituminous substances in like manner are liable to crack at low temperatures. These latter are, however, among the best adapted to the purpose, disposed as a coating with a draft to drain-culverts in the piers by which the water may certainly escape. The most secure mode, however, of protecting bridge arches from percolated water, and preventing it from acting upon the joints of the arch-stones, is to rake the backs of the joints out, and run them full with molten lead, drafting the backs of the stones as before, to lead the water off. In works of minor importance,

horses as soon as the sand is ground to dust by the operation of carriage wheels, whenever water enough is applied to turn the dust into mud. Water alone will not produce slipperiness; and the more such paving is washed and swept the safer it becomes.

and on the backs of brick arches, two or three thicknesses of plain tiles bedded in cement will be found a useful protection to the work.

The road-way upon a bridge should be arranged with an inclination or inclinations of the surface to carry the water off, whether the design of the bridge present a horizontal line or not; and if the length be short,—upon one large arch, or upon three small ones, so that the extent do not exceed 200 or 300 feet,—it will be far better to drain upon the surface through a length from a summit in the middle out over the constructions both ways, rather than to conduct the water down into or among the constructions, where the difficulty of preventing escapes from derangements of one kind and another is very great. Indeed, the water which is found too frequently acting upon the joints in the haunches and springings of bridge arches arises as frequently from escapes from the provided drainage as from filtrations. When the water is permitted to run upon the surface of a road, however, the road-way should be laid with a paving that may act as roof covering as well as paving, and not be itself liable to damage from the action of water upon it. In such a case a broken stone road would not be a proper one, and either shaped stone, or wood, should be laid upon an unyielding bed, and have cemented joints; whilst it may be itself in a condition to receive benefit rather than injury from the water running over it. The roadway being kept properly free from dirt, a fall of from 9 to 12 inches in 100 feet, according to the evenness

of the paving, will relieve it of rain water; and it may take such a fall as this even upon a level bridge without any inconvenience; the lateral footways continuing level longitudinally, and having a similar fall transversely toward the road-way.

Parapets have nothing to do with the construction of a bridge of masonry, but they are required, nevertheless, for the convenient use of ordinary road bridges of whatever substance they may be composed, or in whatever manner constructed. The duty of fencing the passenger from accidental fall over the precipice the bridge forms being all that is required of the parapet, if this be provided for securely, the arrangement as to matter and manner may be referred to the decorative disposition of the works, or what is generally understood by its architectural arrangement.

In proceeding to the execution of a bridge after the design of its constructions has been arranged and determined, the all-important subject of founding the abutments and the piers presents itself. The arrangement of the design, it has been already strongly urged, should be such as to induce the slightest possible additional action upon the bed of a river, so that the process of founding those parts of the constructions that fall within the water-way, or within range of the water-course, may be simplified to the greatest possible degree. Powerful abutments upon the margins of a river are more easily made than piers can be properly founded and built within its water-way; and two piers near the margins are more cheaply procured than four

piers which may be distributed over the whole breadth of the river. For bridges of light weight and great span, whose piers, when piers are required, need produce no sensible effect upon the water-way by inducing no stronger current than the bed has always been subjected to, if that bed be of such consistence that it is not disturbed by the most powerful influence that the river is liable in floods to exert, there should be no objection to founding the required piers in coffers or cassoons;[33] first dredging or forming by concretions a fair and even bed for it within the guide piles that must be driven to assist in mooring and setting the cassoon in which the pier is built. In such a case much will depend upon grounding the floor of the cassoon with an equal bearing over all; for in this, as in all other works of hydraulic architecture, the exclusion of running water from under the constructions is both the most difficult of attainment and the most important to effect. Much too will

[33] This word is commonly written *caisson*, after the French; but as it is not properly a French term, and *caisson* is but the French form of an Italian word, and as we have in English a recognized mode of forming words from Italian without the intervention of French, that mode is used in preference by all who understand the term and the word, and do not affect a French when an English word will express the same idea. Cassoon is from the Italian word *cassa*, a case or chest, with the augmentative *one*,—*cassone*, a large case or coffer; as saloon from *sala*, a hall,—*salone*, a great hall; *palla*, a ball,—*pallone*, a great ball, Anglice,—a balloon. Coffer is common to French and English, and would better express to an English reader what is intended by a cassoon, but that it is appropriated by the dam formed for founding piers by insulating the place upon which a pier is to stand, and laying it dry as a huge coffer.

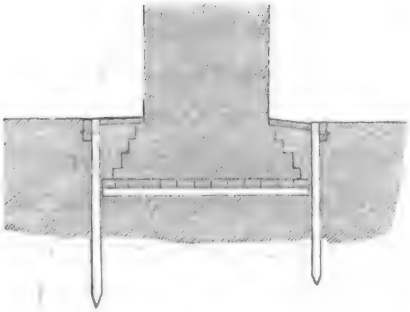
depend upon the pier so founded having enough of its mass in air above high-water level to insure weight to keep it down, seeing that the floor of the cassoon on which the pier is built is of a substance generally lighter than water, and that all but the heaviest kinds of stone have half, and even more than half their weight abstracted upon immersion in water, whilst the upper works are supposed to be so light as not alone to supply the deficiency. If Westminster Bridge had been executed as it is shown at Plate 21, it would probably have been carried away within a very short time, for it could hardly have withstood the effect of a high spring tide ebbing with back water from heavy rains, or in a thaw, with floats of ice, aided by a south-westerly wind. The piers would have been, under such circumstances, entirely immersed, and the current scouring under their timber platforms, but little force would have been sufficient to break the wooden superstructure across in its length of 1000 feet. On such an event occurring the whole would have gone perforce to ruins, and the lighter materials floating down would have formed a rafted dam above Old London Bridge, and so have shortened its duration the better part of a century.[34]

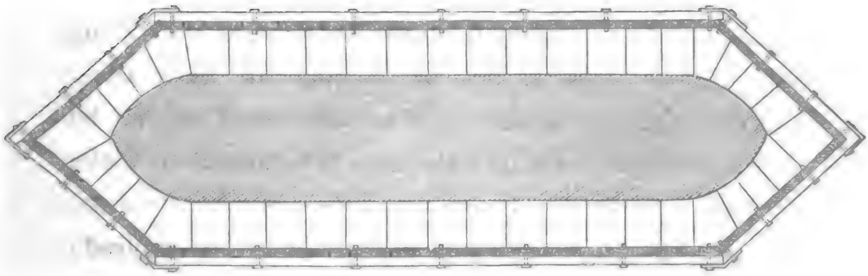
[34] The works in progress at the present time for securing, repairing, and lengthening the piers of Westminster Bridge, have been the means of exhibiting the infirmity of the original design by exposing to view the imperfect nature of the arrangements for excluding the water from under the floors of the cassoons, as well as the imperfect masonry of the constructions themselves. The weight of the masonry superstructure adopted, instead of the timber first intended, gave the work the pre-

When the bed of a river is of clay, or when the clay is covered by only a thin stratum of gravel, close sheet piling may be driven with great accuracy, as the gravel can be dredged out from the line to be occupied around the site of a pier, or rather over the whole site of the piers, and including this line, to form a coffer adapted to the form of the base of a pier. The sheet piling being driven into the clay bed far enough to have a good hold below the depth to which it may be deemed necessary to bed the floor of the cassoon with the pier in it, the ground is to be spoon or scoop-dredged to the required depth, and, the gravel being excluded by the coffer, a perfectly even bed may be obtained on which to ground the platform upon which the pier is built. This platform should be formed of at least two thicknesses of timber traversing one another, laid close to, and trenail-pinned or bolted together, but trenail-pinning ought to be enough. The floor of the cassoon or platform for the constructed pier should not fit or be fitted so closely within the piled coffer as to render it liable to become bound by the piling as it descends, as it may so be prevented from grounding properly, and it may indeed impound the water under it for want of sufficient means of escape around. Nothing can be more important indeed than it is to insure the com-

carious endurance of the last century, by pressing the piers down within the sheet piling driven around them; though, indeed, the celebrated sunken pier sank still further upon the coffer-dam lately made around it being first laid dry, or rather when the pier was laid dry by the withdrawal of the water in which it had been immersed.

plete grounding of the platform on the solid clay, and before filling in over the spreading footings of the pier.





There being means of escape around the platform for the fine mud that is sure to form upon the surface of the clay, even while the cassoon is sinking, as well as for the water that is to be driven out, the pier should be loaded with artificial weight to the extent of what it is intended to carry, to press the platform down upon the ground, and into it as far as the completed superstructure could ever force it. This being done, a curb of iron or of timber, well provided with straps and screw-bolts, should be put round the piled coffer, flush with the ordinary bed of the river, as a waling, or rather as a hoop, for the purpose of holding the piling well together, and of

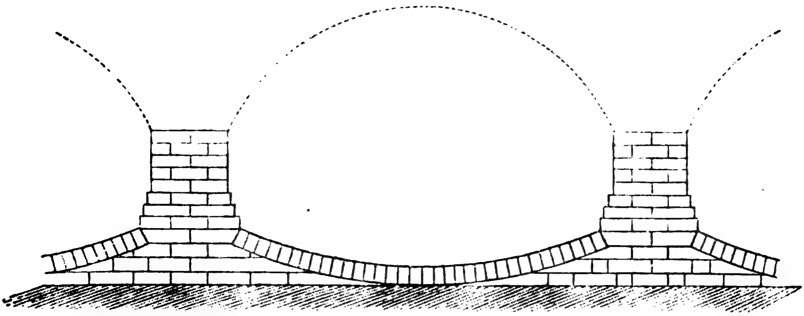
resisting the pressure against it of a stone floor that should be laid at the same level between the body of the pier and the inside of the piled coffer; the spaces under it over the receding footings being first filled in with concrete, and the piles cut off even with the upper side of the curb.[35]

When the bed of a river is sufficiently hard to resist the action of running water, and the scouring effect of water under the greatest pressure to which it may be subjected, or become liable, in the particular case, timber may be wholly avoided in connexion with the permanent construction of stone piers; and the diving bell will give the means of dressing the beds level, and of laying so much of the masons' work as may be under water, where the water is deep enough to make such aid necessary. Upon such a bed as here supposed spreading footings to a pier ought not to be required; but if the substance should be less dense than the stone to be used in the pier, and the pier of the smallest bulk consistent with the execution of its duty with safety, and with regard to its endurance, it will be

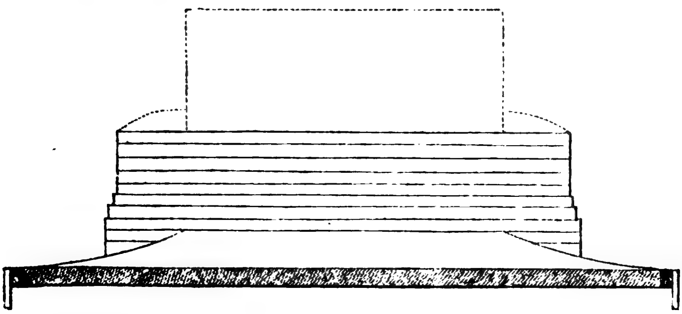
[35] The practice here described is generally that adopted in the works in progress, under the direction of Messrs. Walker and Burges, at Westminster Bridge. These works are carried on with the advantage of coffer-dams, rendering some of the processes above described unnecessary; but as a body of gravel still exists under almost all, if not all, the piers, extraordinary means are required for confining it, and for excluding water from among it; and ample facilities are absolutely necessary, though, indeed, the under-pinning could not have been effected satisfactorily and securely without coffer-dams. A diving bell would be necessary to pave, as proposed in the text, if the water were deep.

better to give the pier greater extent of base than the horizontal section of its body amounts to, even if it be so much more taken out of the water-way, unless it should be deemed the better alternative to hack out the bed of the river, and to place the footings of the pier within it.

The preparation of artificial foundations, whether in the beds of rivers and other waters, or on land, is too large and extensive a subject to ènter into a treatise like the present, but reference may be made here to the remarks on founding bridge piers at page 117 *et seqq.*, *ante*, in connexion with the diagrams at page 118, of the subject of which the following are in further illustration :—



Longitudinal section of a bay, with invert to distribute weight.



Transverse section of bay, showing elevation of a pier and the groining out of invert to sheeted fender-sill.

It ought not to be necessary to mention here the importance in all works of masonry of attending strictly and carefully to that interlacing of stones in courses known by the term bonding, and this in whatever form the stones are disposed; whether in horizontal courses rising vertically, or in courses with beds inclined to the horizon and rising in a bending form, as in arches, or whether it be in the length or the thickness of a piece of work. This may be said, however, that there need be no limit to the length of stones in wrought and gauged work, where the work is accurately wrought and properly disposed by bonding; but if there be any reason in the mode of conducting the work for providing against irregular settlements, stones of greater length than twice and a half the thickness of the courses in which they occur should not be introduced, and generally the settlement of gauged masons' work will be more equable the more regularly the stones are disposed as to length and heading. When masons' work is executed, however, as it ought always to be, and in bridge constructions particularly, so that it shall retain unyieldingly the exact form and position given to it, stones need not be restricted in length in any part, unless it be for the purpose of the bond. In coursed rubble masonry, again, where settlement must be provided against, the stones employed should be in no case of greater length than the proportion above stated; and no stone should occupy in depth more than the thickness of a course.

Cramps and bed-joggles[36] ought to be unnecessary in

[36] The dowel bed-joggle is a substitute for the tenon-like joggle,

bridge piers, unless it be in their cutwaters, perhaps; for good workmanship applied in forming the beds, and good sense in bonding the stones,—proper mortar being applied in all cases, to the perfect exclusion of water from the beds and joints,—will do all that cramps and joggles can do to enable the piers to perform their duty. As cutwaters, however, do not fall directly within the gravitating force of the upper works of a bridge, as the body of the pier does, and as they are liable to severe concussions in a horizontal direction, it is desirable that the upper courses, at the least, of cutwaters should be dowel-joggled in the beds, and cramped in the joints, whilst the heads or covering stones of cutwaters should be in single blocks whenever it is possible to obtain blocks of sufficient size and of proper quality, and the means are at hand of applying them. These should also bond into, and, indeed, form part of the impost or blocking course which receives the springing courses of the arches.

As the abutments of a bridge have to sustain not only directly vertical gravitating force, but also to withstand it in the form of lateral thrust, all those parts of an abutment that fall within the action of lateral thrust should be joggled in the beds; but much labour may

raised at great cost of labour and with waste of material in the heading joints of stones. A piece of slate, 2, 3, or 4 inches square, and from 4 to 6 or 8 inches in length, according to the weight and magnitude of the work, is let into the beds of stones by their angles, and being brought under or over joints, they serve as dowels, joggles, and plugs. A double dove-tail form may be given to these joggles with advantage in some cases.

be saved when horizontal coursing is used, and the mere weight of the mass depended on to resist lateral thrust, by varying the coursing, or bonding in the beds vertically, as well as bonding in the longitudinal direction horizontally. This is a mode of toothing or locking work together, however, that must not be adopted without a proper system; for if it be intrusted to the workman to do it in his own way, either unnecessary expense may be incurred or the work may not be done with the desired effect.

It is very desirable that all the arch-stones of a large and flat arch should be dowel-joggled in the beds; but as the usual dowel-joggle cannot be introduced with the key-course, plugs of proportionate size must be used instead, and the stones may, besides, be cramped together. In arches of small size, or in large ones of quick sweep, joggling may not be so desirable as in those of large size and flat sweep; though it is to be understood that in any case both joggles and cramps should be considered as surplusage, and as precautions merely, to counteract the effect of any imperfections in the work from want of fulness in any of the stones in an arch, or otherwise.[37]

The horizontal coursing in the spandrels and haunch walls, and over the piers, need not be either joggled

[37] In building London Bridge iron bars were let into the back ends or tails of the arch-stones, and run with lead as cramps or transverse ties in several courses, and they do not appear to have produced any injurious effect, though it may be questioned how far they are of any use. They ought not to be of any use.

or cramped, unless they are subjected to any undue or lateral pressure, in which case it is desirable that the various courses should have the connexion that the dowel-joggle gives to secure their acting together. Blocking and cornice courses should be joint-joggled and cramped, and parapet and coping courses joint-joggled and plugged with lead.

In all heavy works of masonry, and particularly in heavy bridge works, to render fine dressing to the external faces of the stones unnecessary, and to carry the bearing at once into the substance of every block employed, it is desirable to take off the arrises at an angle of 45° with the general line of the face of the stone at the outer edges of the beds; and it is particularly important that this should not be omitted to the edges of the beds in the soffits of large arches. The same may be done, as a matter of taste or fancy, in like manner, to the heading joints, but it is not otherwise desirable to them. In setting masons' work, the blocks should be showed into their places, as workmen term it, to see that they are properly fitted and adapted in every respect; and then all the surfaces to be brought together, both beds and joints of the already bedded stones, as well as of the stone to be set, should be first made quite free from dust, then moistened with soft water, and then payed fairly and evenly with the mortar used, the mortar being in a soft but not a fluid state, pressed with the trowel into all the interstices of the stone to drive out the air; and, whenever circumstances will admit of it, in letting the stone to be

set down into its place, it should be made to slide rather than to drop, to allow the air to escape, for otherwise globules may be retained within the mortar, and prevent the joint from being a sound one. Mere dropping down is not enough, indeed, with even the heaviest blocks; but, when seated, the stone should be struck with a mallet, maul, or beetle (of wood), of weight proportionate to the magnitude of the block; for blows may be so applied as to draw a stone up in the joints, while the effect of concussion is to make it settle down in its bed.

In setting the upper courses of a large arch, some have affected to put the stones in their places dry, and to run the joints with grouting mortar to fill them when all the courses and key-stones were placed. This certainly affords the means of arranging the stones very accurately, and of making every one take its proper place and position; but it does not insure the filling of the joints, or the spaces between the stones, with solid matter, nor does it give the means of working it into the pores, if the term may be used, of the stone, unless greater space be allowed in the joints than mortar would occupy if applied in the usual way. Mortars that expand in setting may compensate for the disadvantages cited by occupying all the voids completely; but this quality of expansion may not be sufficient to compensate for the great fluidity that must be given to the mortar to carry it through the fine crevice that alone should be left between the stones of a bridge arch for mortar to occupy. The usual mode in building a large

arch is to bed the stones from the springers upwards on both sides alike; that is, to build up the arch simultaneously on the two sides, or from its two springings, until the space for the key-stone course alone remains unoccupied at the summit of the arch. The key-stones should be worked to fit accurately, but not tightly and so as to bind in dropping down into their places; for if they have to be driven they are likely to force out spalls from the lower angles and soffits of the stones between which they are driven. It is stated in an account of the New or Grosvenor Bridge over the River Dee, at Chester, in the first volume of Transactions of the Institution of Civil Engineers, that, "In setting the key-stones three thin strips of lead were first hung down on each of the stones between which they were to be inserted, and the key-stone being then smeared with a thin greasy putty made of white lead and oil [linseed], was driven down with a small pile engine, the lead acting as a slide, and preventing grating until the stone was quite home." In this case it is not unlikely that the bearing is even now entirely upon the strips of lead, and it would have been much better to use lead sheets to cover the beds on both sides, when the whole of both joints would have been securely stopped, and the bearing distributed certainly over all. A mistaken notion existed probably that some advantage was derivable from the adhesiveness of the putty when it should set; but the putty would shrink in setting, and leave an imperfectly filled joint that no adhesiveness throughout parts of it could compensate for.

The joints of arch-stones of bridges must, in the most accurately executed works, occupy some space, or rather there must be some space between the stones when cementing matter is used at all; and this space should be of equal thickness throughout, that is, the adjacent beds of stones should be in perfectly parallel planes, and these planes parallel to a line at right angles to the tangent of the curve at the soffit of the arch at the point which lies between the stones abutting the joint; or, if the curve be part of a circle, the radius should run through the middle of the space forming the joints, and the beds of the adjacent stones be parallel to it.

Much ingenuity and great expense have been wasted in the composition and erection of centres for bridge arches, and generally the most laboured and the most costly have been the most inefficient. The end sought is the support of the parts of an arch in the exact position assigned them, until the arch is completed and the softer portions of the composition are indurated sufficiently to resist the pressure of the materials of the arch within itself, when the weight may be transferred through the support given to one another by the various parts of an arch, to the piers and other permanent points of support from which the arch is sprung. In pursuing this end, it is absolutely necessary to provide in all cases for the gradual and equable removal of the support upon which the substances employed and arranged in an arch have depended; for although the aim in putting the parts together is, or ought to be, so to dispose, work, and set them, that they may be left

without the artificial support of centering when all the required conditions of the structure are once fulfilled, the withdrawal of the support suddenly or irregularly would occasion a shock that might induce collapse in even the best arranged and most perfectly wrought arch. Besides upholding the parts of an arch before they are placed in a condition to act together for mutual support, provision must be made, therefore, for the equable and gradual, as well as easy, removal of the means used to uphold them, and that such provision should be perfectly under control. The arrangements generally adopted to this effect are shown in Plate 28; and under the completed arch marked F, the method of easing the centering, by driving back the wedges on which it has rested, and of eventually striking it wholly, is indicated by men working pendulum beetles.

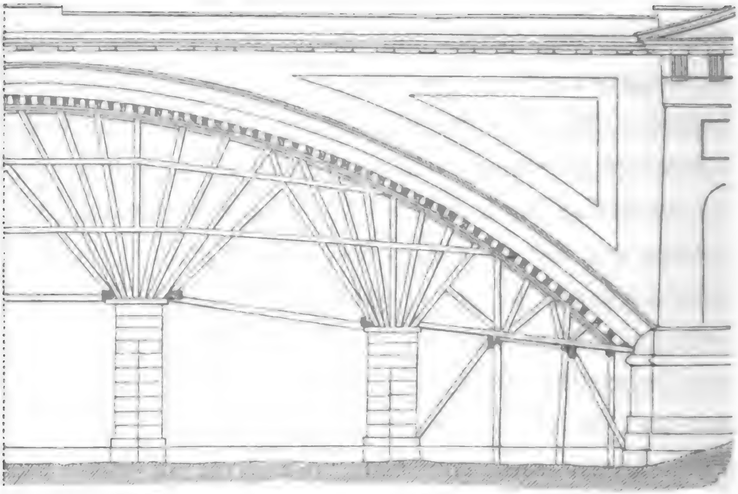
But as the most important bridges are built over navigable rivers, in which it is required that the navigation shall not be stopped by the preparations for building a bridge, the temporary supports to bridge arches, known as centres, have generally to be so disposed, with reference to their own supports, as to leave both breadth and headway sufficient for the passage of craft ordinarily navigating the water-way. To attain this end,—it must be supposed,—centres have been commonly framed in the most extravagant ways, and generally with the effect of making them both bear upon and thrust against the piers on which the arch is to be sprung. The mere bearing of the centering upon the pier destined to carry the arch, upper works, and load, would be harm-

less if it were made to bear upon the body of the pier equably; but the plan generally adopted has been to make the centering, and consequently the arch when in course of construction upon it, to bear upon the footings only, or upon the footings and upon corbels projecting from the faces of the pier. Plates 1, 28, and 52*, show examples of these vicious modes of supporting centres; and all three examples are so composed that they thrust against the points of support laterally, as well as press vertically; whilst they have not the power in a lengthened series, as shown in Plate 28, of counteracting each other's thrust upon the piers respectively, except by virtue of their load, which must therefore be applied throughout the whole length continuously, if not simultaneously. Most of Perronet's centering has this defect; and most of the greater centres by other eminent bridge-builders are open to the objection that the passage for craft is not retained more perfectly than it might be by much simpler means, whilst the centres themselves were far less inflexible than they might be rendered if so much were not attempted.

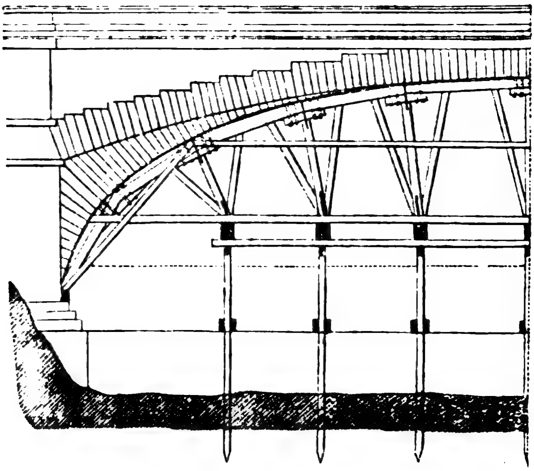
Another very important object, in an economical point of view, is generally aimed at, but mostly without success, because of the wrong direction of the aim. Centering for large arches is considered costly because of the large quantity of timber it consumes; and the attempt to economize in quantity is made for the most part at the expense of almost all the timber employed, because of the cutting and boring to which it is subjected to

combine it,—at the expense of large quantities of iron-work to step, bind, and tie the timber,—and of labour in cutting, boring, fitting, and putting together, that might be dispensed with in proportion as these operations are limited. Now the proper mode to economize in bridge centres is to arrange the timber in such manner that it may receive the least possible damage from cutting and boring, and be of use again. Smeaton professed to compose his centering upon this principle, not so much to employ little timber as to dispose what he used so as to injure it as little as possible, and thereby economize not only in timber but in iron-work and in labour. Piles driven into the bed of a river in bays wide enough for craft to pass through, and sufficiently deep to carry the weight to be imposed without yielding, are more efficient for the support of the load than the remote footings of the piers can be, when the weight has to travel to the latter through the ramified thrusts and bearings of a large framed centre without a longitudinal tie. In this point of view, the mode of supporting the centering shown in Plates 25 and 85 is far preferable to the modes indicated in Plates 28 and 52[a]. Two of the best examples published, however, of bridge arch centres,—for economical composition, proper disposition to carry the load of an arch in course of construction, and for efficient supports,—owe nothing to the engineer-architects who designed the bridges, or directed their execution, but are said to be the productions of the contractors who had undertaken the works. These are the centres of the great Chester Bridge by Mr. Trubshaw, and

of Mr. Telford's Gloucester Over-bridge by Mr. Cargill,



Mr. Trubshaw's centering to Grosvenor Bridge, Chester.



Mr. Cargill's centering to the Gloucester Over-bridge.

the respective contractors for executing the works of those bridges. The duty of the temporary stone piers used by Mr. Trubshaw must be performed in deeper

water than is here indicated, by piles, and piles more frequently repeated than the piers are, or driven in couples to receive the radiating struts, and it is to be understood that the wedges, by means of which the bearing of the arch is to be taken off the centres, are under the lagging or open planking on which the stones rest, and not under the centres themselves; so that the easing may take place in the most gradual manner, and the lagging may be taken out plank by plank, and from any part that may be desired first, as the work may give indications on being eased. A point greatly deserving attention in this centre is that the bearing timbers are all opposed to the weight endwise, so that the effect so commonly and mischievously felt in the common practice from the shrinking of timbers in their transverse sections, is avoided.

Mr. Cargill's centre is already adapted for deep water, and may, perhaps, be considered the more workmanlike centre of the two. It has much of the advantage possessed by the other in the principal timbers being made to act as props or shores; but as the wedges for easing and striking are placed under the centre to let it down bodily, there are of necessity some important timbers bearing in the direction of their thickness.

"Independently," says Perronet, "of the choice of materials, of the exactitude of the arrangement, and of the care with which the stones should be wrought and set, the success of these great arches depends essentially upon the centering employed, and upon the means used of setting and striking it. For want of giving suf-

ficient attention to this, it has often happened that the forms of the arches have been deranged, and some arches have actually fallen: these considerations, which have reference to works of the greatest importance, have appeared to me to merit the attention of the Academy and of the public, as well as of those who are intrusted with the duty of designing and constructing them."[38]

[38] *Lu à la rentrée publique de l'Académie Royale des Sciences, le* 21 *Avril,* 1773.

ARCHITECTURAL TREATISE.

IF a work as a bridge be well composed constructively, whatever may be the constituent material or materials employed, and whatever may be the kind of construction, it can hardly fail to be an agreeable object, for it will certainly possess the essentials to beauty in architectural composition,—simplicity and harmony. The introduction of any thing not necessary to the construction, the omission of what is requisite, or the substitution of a bad expedient for a good one, will assuredly tell injuriously upon the eye, how incompetent soever the observer may be to determine the cause of the defect, or even in what the defect may consist. It is impossible, therefore, to draw any line between the constructive and the decorative,—or what is commonly termed the architectural,—composition of a bridge; but it may not be without use to point out what is to be avoided, and to suggest how the works of a bridge may be at the same time economized and made consistent with a better taste than has been generally brought to bear upon such compositions.

Let the designer dispossess his mind of any preconceived notion that a bridge should be of such and such forms and arrangements, and aim simply to supply a construction to fulfil the services required with the means and materials at his disposal; and he should also most carefully avoid all reference to what he may have understood to be necessary to architectural adornment, unless such should grow out of the subject itself and become essential to it. Brick and stone, when used in bridge constructions, are almost of necessity cast into the form of an arch or of arches, because these substances are most conveniently applied in that form to fulfil the conditions required of a bridge; but although timber and iron may be applied in bowed forms with great advantage in some cases, neither of these substances can be employed in practice for the construction of arches without inconsistency, nor can the forms which brick and stone almost impose upon the constructions in which they are employed, respectively or combinedly, be produced with either timber or iron without absurdity. It is not more absurd, however, to compel timber and iron to assume forms inconsistent with their qualities and capabilities, than it is to apply to a bridge devices, with a view to decoration, that may have been found, or fancied, productive of good effect in, or upon, a palace or a church.

It is false taste that seeks to compel materials to assume inappropriate forms,—forms inconsistent with their constructive qualities,—and that imposes arrangements for decorating a bridge as capricious as attempts

to improve nature by carving an oak into the form of a yew, or by clipping a yew into the shape of a peacock. There is an almost inherent source of false taste in bridge elevations, found in the practice of the middle ages, when bridges were made for men and horses only, and consequently very narrow. This consists in carrying up the cutwaters as buttress-counterforts to the piers the whole height of a bridge, to enable the piers to withstand the weight and flow of water in floods, and which the combined shortness and thickness of the piers, and the comparative diminutiveness of the bays, of bridges rendered constructively necessary. A bridge wide enough for wheeled carriages to pass upon it, or, indeed, any bridge properly composed as to the relative thicknesses of the piers and the widths of the openings between them,—the work being withal properly executed,—can never require extraneous strength to resist the force of the water in ordinary cases; but notwithstanding this, in situations where nothing in any degree extraordinary exists, or is likely at any time to arise, Westminster and London Bridges have counterfort projections against their faces over the cutwaters, and Blackfriars' and Waterloo Bridges have distorted representations of parts of an ancient Greek or Roman temple, or other structure of the kind, appended to them, by way of giving an *architectural* effect to the elevations, whilst it is certain that they are as inconsistent with good taste, as their uselessness in the constructions shows them to be with good sense. It may be remarked, in continuation, that in the bridges of the middle ages the upper ter-

minations of the cutwater counterforts upon the ends of piers were turned to use, as they formed recesses that practically widened the road-ways by affording spaces at intervals in which travellers could stand aside while a pack or other laden horse or mule passed by. In modern practice these recesses have still been made, though the excuse for them has ceased with the increased width given to a bridge, especially when there are foot-ways clear of the carriage road, and the use to which such recesses are too frequently applied renders them, not only unfit for resting-places in which to stand and look around, but offensive to the decent passer by.

The piles or piers, as the case may be, must of course have in themselves, and with their cutwaters, length enough in the direction of the stream to withstand the greatest force of the water, of any thing that the stream may bring along with it, and of the wind acting in the direction of the current of water upon the superstructure. This length, in the case of a foot-bridge whose upper works are timber or iron, may be of necessity much greater than the width of the bridge; and, to allow the bays, or openings from pier to pier, to be as great as possible, the main bridging timbers or iron frames, as longitudinal beams or ribs, require to be kept the full width of the intended road-way apart, to obtain sufficient effect for the horizontal bracing of the beams or ribs to stiffen them and check vibration; but when the road-way is intended for carriages it must be so wide that the length of the pier may be generally brought within the width

of the bridge. In most cases, however, when a bridge exceeds in width one-fifth of the longitudinal bay or span from pier to pier, the main bearing constructions may be restricted to that proportion, unless that should fall within the extent of the heavy and concussive traffic; and the floor may be made to run out, or oversail, laterally, with its joists or transverse bearers, as the arrangement may happen, the whole or any part of the extent of foot-ways. A bold effect can be thus produced by an economical arrangement of the main constructions, and any requisite additional stiffness to the projected parts of the floor may be given by stepping straight or curved struts or solid brackets into the longitudinal beam, or other under-works, upon which the ends of the joists may be again secured. If the situation and use of the bridge render enrichment desirable, the oversailing ends of the joists and the brackets may be cut or cast of moulded forms; these being capped by the edges of the floor rendered deeper by the plinth of a parapet, and a well-disposed parapet railing being carried out to the extremity to crown the work, will give decorative effect to the fullest extent that can be required.

Most of the larger timber and iron road bridges hitherto erected might have been economized to the extent of one bay, or of one set of ribs or main longitudinal beams, at the least, without detriment to their stability, and, in many cases, to the great improvement of their appearance; or, inversely, the effective road-ways of such bridges might have been increased in width at a very trifling cost. One set of ribs and a length of the piers

equal to one transverse bay of Southwark Bridge might have been omitted, and a great saving effected in the cost of the whole work; or the efficiency of the bridge might have been increased and its appearance improved at a comparatively trifling additional expense: the same remark will apply to almost all similar constructions of materials that may be trusted to carry weight transversely, as timber and iron may.

The upper works of the Pont d'Ivry, a timber bridge over the Seine near Paris, on piers of masonry, is very much in the manner here recommended, and with very good effect, but that the parapet railing does not stand on the outer edge to give full value to the bold oversailing cornice, both as a matter of decorative disposition, and to bring the space within the road-way. The effective width of the bridge might have been 3 feet more than it is, and the appearance of the work greatly improved, with no addition to the expense.

When the constructive arrangement for bearing is such as to require the framing to rise above the level of the road-way, this can be only effected without inconvenience at the extremities of the width, or between the carriage-way and the foot-ways. The disposition above indicated would place the rising ribs in the latter position, and between the two positions this is constructively the better. Railway bridges require the rising ribs to be placed at the extremities, of necessity, because it is not supposed that more width is given on such bridges than enough to allow the trains to pass freely and leave room for a man to stand in safety against the parapet.

As a matter of taste, as well as of constructive propriety, the masonry or brick-work piers of bridges, whose carrying constructions and upper works are of timber or iron, should terminate where the carrying constructions commence. The iron bridge over the Lary, near Plymouth, before spoken of in a note at page 78, presents an example of much good taste in this respect, as in respect of the form given to the iron ribs, the piers presenting throughout a fair substructure to the bridging works, whilst they are themselves well composed and of graceful form.[1] Southwark and Vauxhall Bridges, over the Thames at London, have the masonry unnecessarily, if not injuriously, carried up between the bridging works; and, in like manner, the timber viaducts upon the Newcastle, North Shields, and Tynemouth Railway, (Plates 11, 12, 13, 14, 15,) in many respects admirable compositions, are open to objection. Far less substance in the piers themselves would have been sufficient for useful service and for appearance, if the masonry had ceased

[1] In this work, however, something might have been saved in the width of the bridge-works, and at the same time greater width of road-way obtained, by making the cornice an effective part of the floor,—the cornice being corbelled or bracketed with somewhat more boldness than at present,—and the parapet railing being placed at the outer edge, adding thereby the projection of the cornice to the available road-way, and greatly improving the appearance of the bridge by the effect of a massively edged platform for the substructions to carry. The Whispering Gallery at St. Paul's in London is formed, in the manner here proposed for widening the road-way of a bridge, on the back of a cornice; the parapet railing being placed nearly on the edge, and the cornice being supported by a consoled or corbelled blocking in the frieze of the entablature to which the cornice belongs.

where the arch-formed ribs spring, and the necessity for a useless rubble or concrete core avoided. It would be hypercritical to object that the dripstone cornices belong to another style than that to which the segmental ribs appertain, as they are well adapted to their purpose; but there needed no set-off on the faces of the piers between the base moulding and the springing level. The transverse section of a pier, longitudinally of the bridge, at Plate 13, shows a much better outline than the same pier presents in a longitudinal section, (called on the Plate the " transverse section,") where the dripstone cornice on the buttress faces breaks the sober simplicity which the outline of a bridge pier should maintain.

The Pont d'Ivry, before cited for the bold cornice-like effect produced by the oversailing of the bridge-floor at the sides, has the masonry carried up through the parapets with most mischievous effect upon the carpentry particularly, and with great injury to the general appearance of the bridge. Indeed, this feature of the work, and the absence of what the French themselves call *pose* to the ribs generally, give the work a temporary effect, whilst the pendant ends of the radiating ties are in the way, so as to be most inconvenient for laden craft navigating the river.

Mr. Stevenson's Timber Bridge, over the Clyde at Glasgow (Plates 6, 7, 8, and 9), may be pointed out as an example of a plain unaffected piece of composition, which only requires the bolt-heads and nuts to the walings to be sunk,—and the saving in the length of the bolts would have paid for the labour of sinking and

stopping over them,—the arrises taken off the outer edges of the waling pieces, where they are exposed, and the monotony of the parapet railing relieved by the substitution of a simple cross rail, as in the otherwise very ugly foot-bridge at Abbey St. Bathans (Plate 16), or some such expedient, and all that can be required in a simple composition of the kind would be attained. Perhaps, however, it might have been improved in effect, as well as economized, by the retrenchment of one row of piles, and the distribution of the rest under the floor; or perhaps two rows might be spared in such another case, widening the transverse bays in proportion, and bracing the horizontal bearers of the bridge floor in the very judicious manner adopted in Mr. Bull's occupation bridge at Plate 10.

The composition of the abutments of a bridge, of whatever materials the carrying constructions of the bridge may be, should be bold and simple, of few parts, and standing well back from the abutting works. The abutments of Staines Bridge (Plate 53) are of this character; and such are the abutments of Perronet's intended Melun Bridges (facing page 37, *ante*), whilst the abutments of Darlaston Bridge (Plate 108), of Gloucester Over-bridge (page 39, *ante*), and of the bridge over the Dora at Turin (page 42, *ante*), are thrust close up to the abutting arches, and the two former, and the Chester Bridge (page 44, *ante*), are deformed with small and mean parts, as pilasters and niches, or other things of the kind. The abutments of Telford's iron bridge over the Severn at Tewkesbury, spanning 170 feet, are formed by

a series of small pointed arches of less than 5 feet opening, forming a bad composition of their style, and weak adjuncts to the massive curved beams that lie between them. The abutments of London Bridge would be very fine, if they stood back clear of the springings of the abutting arches; and both the granite arches of London Bridge and the iron arched-beams of Southwark Bridge appear to want something to rest upon as well as to abut against. This is, indeed, a defect so common that a student might suppose it to be almost essential to the composition of a bridge, that the arches should appear to hang between the piers and abutments by virtue of their lateral thrust or tendency to spread, rather than to require or to receive any support in virtue of their more natural direct gravitating force acting upon well-defined bearing points. This is a defect not to be found, however, in the remains of the bridges of Roman construction, which are, nevertheless, the type of the modern form in almost all other defective points. The abutments of the bridge at Neuilly are well composed, but like the rest, the arches are squeezed in between, rather than placed upon, the points of support, whether they are the piers in the water-way, or the half piers on the faces of the abutments. Westminster and Blackfriars' Bridges, though both overlaid with pseudo-architectural decoration, are without abutments as to appearance in their elevations, and are consequently defective in a very important characteristic, and one that is quite essential to give full effect to the best arrangements as to composition in other respects. So important are abutment

compartments in the composition of a bridge as a matter of taste, as well as of construction, that it will be often worth while to take something more from the water-way, even if it be between retaining walls within a town, than to let a bridge stand in a cramped manner, as the bridge of the Champ de Mars does (Plate 56), and as the Wellesley Bridge at Limerick (Plate 54) appears to stand, between the quay walls on either side, as the inner or under arches of the latter do between the piers.

In displaying the abutments of a bridge, lines that may have the effect of lengthening the bridge itself in appearance are better than those which bring out the abutments to the disadvantage of the main part of the work. Lines extended in the direction of the bridge itself are perhaps the most appropriate; but as bridges have to be joined on to quays or to approaches variously arranged, curved lines may become necessary; and in this case they should be concave on the outer face, both for the composition and for convenience. The Italian bridge at page 42, *ante*, presents a wretched composition in every respect, but in nothing is it worse than in the convex masses into which the abutments are thrown.

The half-elevation of a bay, corresponding in general form and dimensions with the collateral half-elevation of a bay according to the existing design of London Bridge, in Plate 39, is intended to show, with other things, that a pier may be reduced in bulk with reference to the water-way, without diminishing the extent of its base or its efficiency, even in appearance, whilst it is believed that the form given to it is much more graceful than the

form of the existing pier, and that the full developement of the arch from over the head of the pier above high water level improves the effect of the composition as a matter of taste, as it certainly diminishes the injurious effect of such a work as a bridge upon the water-way. The piers of London Bridge, from the footings up to the level of half tide, where the arches are sprung, are in thickness about one-sixth the reach of the two half arches resting upon them respectively, but at the level of Trinity high water, the space occupied by a pier and the immersed springings of the arches has increased so much as to exceed one-fifth the two half arches; whereas by the arrangement proposed, starting from the same extent of base, and taking thereby all that is essential to strength in the foundations, and with ample substance still remaining in the body of the pier, the average space occupied by the pier is but one-seventh of the void, whilst at the springing level assumed, being that of Trinity high water, the solid is reduced to one-eighth the void, instead of being increased as at present from one-sixth to one-fifth between half tide and high tide. It has been already shown that piers may be greatly reduced below the proportion of thickness of the piers of London Bridge; and the reduction here proposed leaves the work still with so much strength, or power of sustaining the weight of the superstructure, in excess, that it is further proposed, as shown in the right-hand half of the longitudinal, and in the transverse sections, to abstract from the bearings of the upper works over the piers, and to introduce a longitudinal groining, as first

suggested at page 166, *ante*. This arrangement would allow the piers to be still further reduced in bulk by turning an invert within the length of the pier equal to the span of the groining, as shown by the dotted lines on the transverse section; but as this would interfere with and disturb the current already divided by the cut-water, the faces of the pier are preserved unbroken and carried up with gauged masonry throughout; the heart being filled in with merely coursed rubble where the invert would occur. The longitudinal groined vault is proposed for economy, and not on account of any decorative effect it might produce upon the composition; though it is quite certain that, seen from the river, and in passing under the bridge, the effect of the long central groined vaulting would be very striking.

In the diagrams last alluded to, and in the right-hand half-elevation on the same Plate, it will be seen that the arch-stones have been reduced in length, as it has been shown they might be reduced with safety, whereby the head-way under the bridge might be increased without raising the road; but as the head-way may be supposed to be enough as it is, the new design is made to coincide with the present work at the soffit of the key-stone course, and the advantage gained in reducing the thickness of the arch is given to the road, making it thus nearly 2 feet lower over all. Under the circumstances of the locality in which London Bridge was placed, a reduction in height of the road-way on the bridge to this extent would have given the means of saving a large expenditure, while the ascent to the bridge on the south side would have been so much less steep.

The collateral half-elevations of a bay and the adjacent transverse sections of the present bridge, and of the suggested improvement of the design of the same, show that it is proposed to alter the caps or copings of the cutwaters in such manner as to accord with the remarks before stated upon the composition of this part of a bridge elevation. The counterfort or pilaster projections are omitted as useless, and the recesses upon them are rejected as inconvenient. The caps of the cutwaters are proposed to be of massive blocks, forming the extremities of the course upon which the arches rest, sunk to weather in curved forms, which are thought to yield a more graceful and fitting termination than the present blocking course with the convex blocks lying upon them and against the springing and haunch-stones of the arches.

However well, or however ill, the abutments, piers, and arches of a bridge may be composed for decorative effect, the best arrangement of these parts may be rendered ineffective, and the worst may be subdued, by the mode in which the elevations of the faces are crowned. The elevations of the bridge of the Champ de Mars at Paris (Plates 56, 57), and of the bridge at Staines (Plate 53), have been already spoken of as examples worthy of notice; but it is believed that both these might be greatly improved by a more effective arrangement of the parapets and cornices, though they are of greatly superior character in these works to the corresponding parts of works of the kind generally. Depth and projection are desirable in all external cornices; in ordinary works of architecture, as houses and other roofed erections, a cornice is the oversailing of the roofs

to cover and protect the walls of the building, as the roof does the building itself, and in such manner that the cornice may form the bed or cushion on which the roof rests. A bridge requires no roof, and a cornice disposed like the cornice under the eaves of a roof is an absurdity, as a bridge parapet is on the top of a house. The common modes of design for the two classes of structure make an interchange of incongruities; the bridge parapet, whether well or ill composed as such, is put upon houses to spoil the effect of the cornice which should be formed under the seating of their roofs; and the weathered cornice which a roof projects over a house is placed upon the faces of a bridge, whilst the parapet remains a distinct feature, and is as if the cornice were not there at all. Now, the aim of the bridge designer should be to obtain the greatest width of road-way with the smallest extent of substructure, as before shown when speaking of timber and iron bridging works; but as masonry is composed of a material that will not carry, or rather that may not be trusted to bear or carry over, unless it be with great substance, the road-way of a bridge of masonry must be kept within, or it must not greatly exceed, the extent in width of the substructure. The whole extent in width may be taken, however, as the dead weight of the parapet may be trusted to oversail its own thickness at least, and to rest on corbels or other arrangements of stone, of depth enough to carry all that can thus be imposed upon them; though it may not be advisable under all circumstances to trust any part of the motive and concussive load to act beyond the extent of the main structure: with proper con-

structive arrangements, nevertheless, and under favourable circumstances, the parapets of a bridge may be set over even much more than the proportion indicated.

The mode here proposed of widening the effective working surface of a bridge was adopted on the flat roofs of the castles of the middle ages, where space was obtained for men to stand, often greatly exceeding the clear surface of the structure, by setting the parapets out on bold and massive bracketings or corbellings;[2] the parapet and corbelling forming together a cornice having such depth and projection as to give the works in which such an arrangement occurs, a character of grandeur and dignity which they retain though in ruins. Taking examples from the illustrations of the present work, the road-way of the bridge of the Champ de Mars (Plates 56, 57) might be thus made 4 feet wider; or, the road-way being the same, the structure of the bridge might be 4 feet less in width, with very great advantage to the composition of its faces for effect, as well as to the funds at the architect's disposal. The parapets are now mere blocking courses over the cornice, and the work is so arranged as to make the parapets appear necessary to keep the stones under them from toppling over (see transverse sections of the bridge, Plate 57). Suppose the bracket-block course, or indeed the bracket blocks alone if got out in bars of a strong stone, run through the extent of the foot-ways,—abstract the moulded part of the cornice, saving both labour and material, and set

[2] The original intention appears to have been to obtain a species of vertical loop-holes through which to hurl stones down upon or to discharge arrows at assailants.

the parapet out upon the cornice to oversail the face of the corona just enough to receive a throat and to relieve the depth of plain face, and put dovetailed gun-metal joggles in the beds to secure the parapet;—all the effective parts of the present cornice would be preserved, but the cornice would be changed from a merely blocked shelf to a deep massive crown to the work, and the road-way would be widened the thickness of the parapet on each side.

By corbelling out the parapet with or without blocks, so that there is a sufficient length of the projected bearing stones upon the work, and sufficient depth in the moulded bed to make the construction secure, Staines Bridge (Plate 53) might in like manner be made effectively wider, and be greatly improved in appearance at a very trifling expense; though indeed all that is here intended might be done in the original construction of a bridge so as to produce a great abatement in the expense, or to obtain a greater width at the same cost. In Wellesley Bridge at Limerick (Plates 54, 55), the moulded archivolt should have been sunk below the faces of the spandrels, and the bed mould of the cornice would then have projected over it with greatly improved effect; the mouldings themselves are susceptible of much improvement, and half the labour and materials employed in and upon the gouty balusters and their over-wrought and inconveniently placed coping[3] would

[3] This coping is placed, with reference to the foot-ways of the bridge, at the level of the eye, confining the attention of the passenger to the *tasteful* work of the architect, and in effect contracting the width

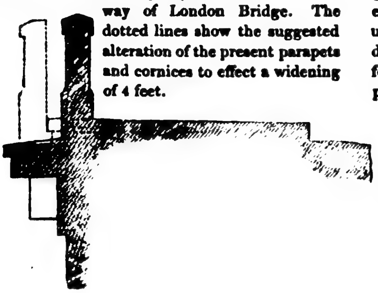
have sufficed to extend the bed-mould course the whole width of the pavement, or indeed to form the pavement,[4] and to raise upon it a bold and richly moulded parapet,—of such height, however, that it might be truly a parapet, or breast high, and not an obstruction to both head and eye, as the present parapet must be. With these modifications alone the road-way might be increased 2 feet at the least.

London Bridge might have been made of its present width as to road-way, by an effective system of corbelling, without the addition to the substructure which was made at a cost of £42,000, and the character of the elevations at the same time very greatly improved. The corbelled parapet forming a deep and boldly projecting, and therefore effective cornice, as shown in the right-hand half elevation, and as indicated in the right-hand transverse section on Plate 39,[5] and as further illustrated by the following diagrams, will serve to make the suggestion intelligible.

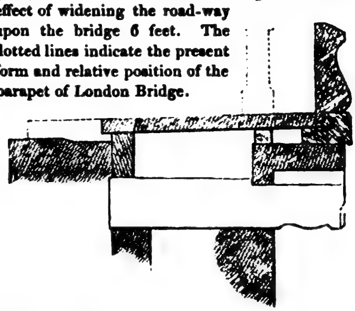
of the bridge by jutting out at the level of the head where most room is wanted.

[4] This being slight, should be of a strong kind of stone, such as the neighbouring Valentia marble.

[5] Mr. Telford seems to have imagined a cornice somewhat in the manner proposed, for Tongueland Bridge; but the idea is quite lost upon the bridge itself, the oversailing of the parapet being confined to certain semi-cylindrical projections upon the abutments which are to be considered castellated. Over the arch the parapets are set in according to the usual practice, and a little mean shelf is all that is left of cornice over some heavy corbel or bracket blocks. To add absurdity to ugliness, the parapets are machicolated, or, as Mr. Telford expresses its peculiarities, "the bridge is turreted and embattled."

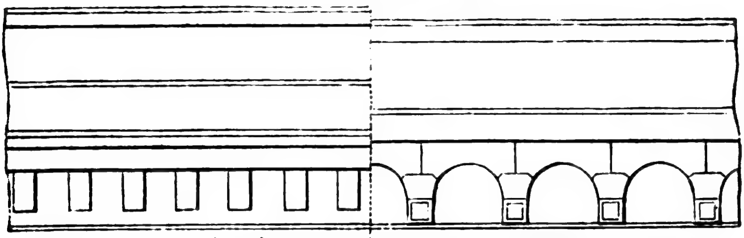


Transverse section of parapet, cornice, and foot-way of London Bridge. The dotted lines show the suggested alteration of the present parapets and cornices to effect a widening of 4 feet.



Transverse section of a corbelled and moulded parapet adapted to London Bridge, with the effect of widening the road-way upon the bridge 6 feet. The dotted lines indicate the present form and relative position of the parapet of London Bridge.

Scale ⅛th of an inch to a foot.



Elevation of present cornice and parapet of London Bridge.

Elevation of suggested corbelled parapet adapted to London Bridge.

These diagrams show the manner in which the work may be composed constructively; and, as to decoration, either plainly corbelled, or with moulded and enriched faces to the parapet, in addition to the corbellings. It is to be remarked that a tongue may be raised on the beds of the stones forming the parapet, 2 or 3 inches deep, as shown in the right-hand section, or a back rabbet may be left in the rough to receive the landings for paving the foot-ways, as indicated in the left-hand section. These give the means of securing the work more effectually than joggles or plugs, and by such simple arrangements London Bridge might have been made, at no sensible increase of cost, 3 feet wider on each

side, amounting to the addition that was made at the before-mentioned cost of £42,000.

A technical eye having regard to economy might object to the quantity of sunk and moulded work which the more decorated of the two diagrams exhibits; but if the arrangement be analysed, and the construction supposed original instead of an alteration, it will be found that the labour involved hardly exceeds that expended upon the present cornice and parapet of London Bridge. The one deeply sunk and moulded face of the parapet will hardly exceed the two sunk faces of the present parapet; the fillet projecting over the great cornice of the present arrangement, with the weathering, would more than pay for the tongue, and the back rabbets to receive it, of the moulded parapet; and the cost of raising the blocks under the present cornice, as they are raised, in the solid, would nearly counterbalance all the work upon the corbels and corbelling arches. The main difference would be in the materials, but a greater quantity of material is absolutely necessary under all circumstances to a greater surface, and in this case it is only the superficial work to the greater extent of road-way obtained that is in excess.[6]

[6] The addition of 4 feet might be made to the effective width of the road-way upon London Bridge at any time at a very trifling expense, and without interfering with the use of the bridge for a day, as the foot-ways only would have to be disturbed, and these one at a time. The expense of the greater improvement to the usefulness and to the appearance of the bridge, shown on the right-hand side of the last page, would be but trifling compared with the cost of adding to the width of the road-way of such a bridge in the usual manner.

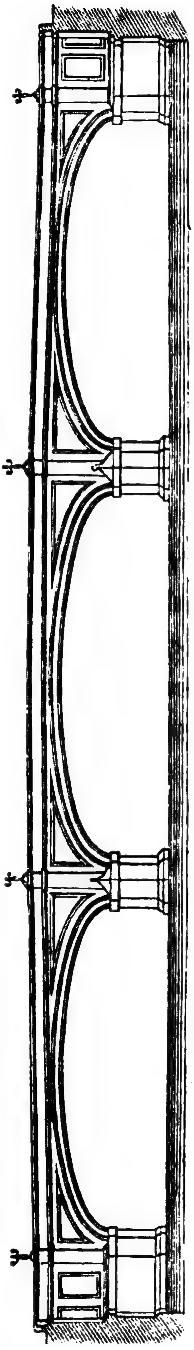
It may be imagined, perhaps, that the style of composition here suggested would be inappropriate except with the pointed arch, but such is not the case. The arrangement admits of a horizontal disposition, and has not necessarily any association with the pointed arch. Indeed, the castellated structures which present the arrangement referred to are rather of the Gothic or round arch period, than of the pointed, and certainly it composes and associates as well with the simply circular as with the pointed arch.

An illustration of this may be seen in the Temple Church at London. A bracketed or slightly corbelled parapet crowns the elevation of the early horizontally disposed circular western part of the church, and the same is carried along the flanks of the body of the church or chancel, which are otherwise composed in the vertical or pointed style, and it will be found to consort well with both. This is an example from ecclesiastical architecture; but it is in the truly Gothic castle at Spoleto, and in many other similar structures in Italy, that the corbelled parapet is carried to the greatest extent of projection, and in which the advantages derivable from it for both use and decoration are most striking. France, Germany, and England have many illustrations of the corbelled parapet; but perhaps the best finished and

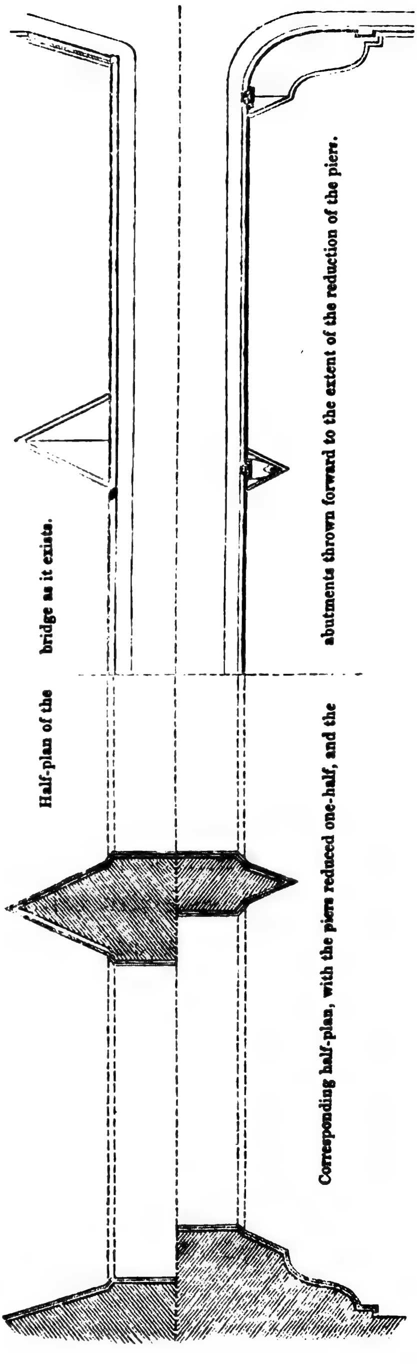
Mr. Telford widened an old bridge at Glasgow by means of a sort of outrigging perched upon the cutwaters, and he carried the idea to Edinburgh and there embodied it upon the Dean Bridge: corbelling would have been better in both cases: the old bridge was made grotesque by the additions, and the new bridge is very far from presenting an improved style of bridge elevation.

most effective example in England is in a comparatively recent work, Thornbury Castle in Gloucestershire, of the time of Henry the Eighth.

Taking a work of less bold but of more decorated character than London Bridge, and to avoid the monotony that would result from the constant repetition of corbelled parapets, as of the shelf-like cornices and frittered balustraded parapets that have hitherto prevailed, the whole face of a bridge may be projected upon a deep and boldly moulded archivolt, as in the Trinità Bridge. This work will, indeed, form a good example by which to illustrate in a varied form what has been said as to the composition of abutments, and to show that huge piers are not essential to the beauty which the Florentine Bridge is generally thought to possess. The topmost diagram on the opposite page exhibits an elevation of this bridge as it is, reaching up to the quay walls on both sides, with its ponderous piers having counterforts built up on the cutwaters to the level of the road-way, but not opened to it, as the practice has been with such in England, so that there can be no pretence that they are of use unless it be as counterforts, and with enwreathed escutcheons on the parapets resting on carved key-stones. The middle figure is an elevation of the same arches, similarly moulded on the archivolts, and having panelled spandrels as in the existing work; but the piers reduced one-half, and the space obtained by reducing them thrown into abutments, composed in the style of the bridge itself, and placed between the quay walls and the bridge works, in such a manner as to give



Elevation of a similar design, giving the same water-way, with piers reduced and abutments projected.



Half-plan of the bridge as it exists.

Corresponding half-plan, with the piers reduced one-half, and the abutments thrown forward to the extent of the reduction of the piers.

much greater effect to the composition. Instead of counterforts over the cutwaters, slight pilaster-like projections are made to form plinths for candelabra, and the acutely formed angular cutwaters are terminated by the representation of the prows or rostra of galleys, which may be in stone or metal according to circumstances. On the pent-up Arno at Florence these might be of marble, and on the free and well-worked Thames, of gun-metal. Such antefixæ would not be for decoration merely; they would serve in floods to prevent drift from being thrown up on the cutwaters, and thus assist in keeping the channels clear, as well as in protecting the exposed arrises of the lower arch-stones from injury. Such a modification of the bridge of the Trinità extended to five arches, and the arches increased in span to produce the required length, might with great advantage replace either of those disgraceful piles known as Battersea and Putney Bridges upon the Thames, near London.

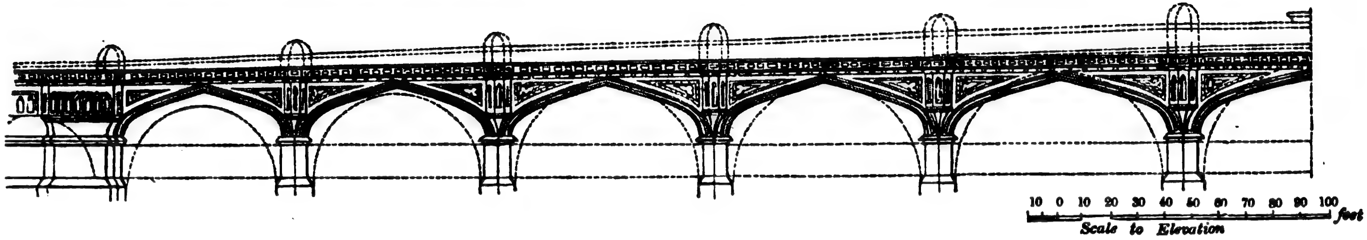
It is difficult to close a Treatise on Bridge Architecture without remarking the increased unfitness of the present superstructure of Westminster Bridge. The arches spring at a level very little above that of low water, where the tide rises and falls from 15 to 18 feet, so that, as before remarked,[7] "the water-way is nearly 50 feet, or about one-sixteenth, less at the height of ordinary spring tides than at the level of low water in the river." The arches contract the way for navigation

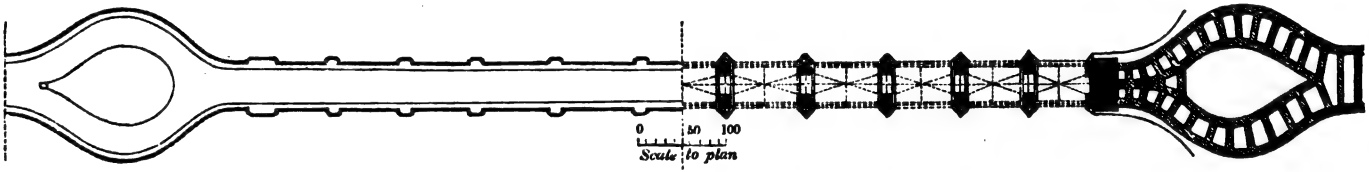
[7] *Ante*, p. 139.

much more than it is at all necessary they should, even upon the present piers; and there is more than twice the height from the soffits of the arches to the level of the road-way than there need be; the parapets are alike offensive by their great height from the road-way and by their ugliness in detail, and injurious by the draughts induced by the perforations of the balustrades; and the solid counterfort buttresses over the cutwaters with their inclosed and cupoletted heads, add needlessly to the weight upon the piers. The bridge is unfortunately near to the magnificent buildings of the Houses of Parliament, and its great height renders this proximity more injurious than it might otherwise be. In all probability some abatement will be made of the height of the bridge in the process of the works now (1842) in hand for securing the piers, and doubtlessly the same good sense which opened a view of the river from Blackfriars' Bridge will open the magnificent prospect Westminster Bridge can command, by substituting parapets, which shall be truly so, for the perforated walls which now hedge in the road-way; but the arches will still continue to render the navigable water-way narrower and more inconvenient than even the multiplicity and thickness of the piers, or the condition of the work, impose. The character of the work, too, will still remain inconsistent with its position at Westminster. It ought, therefore, to be completely remodelled. As the piers are now in process of being repaired and secured, and so as to be free from any danger, founding new piers is out of the question, and the piers cannot be reduced

in number without imposing additional weight on those which may be left; a condition which the original defective founding, and the badness of the original structure, forbid. The whole of the superstructure might be removed, however, and the piers being carried up from the level of the present springing to that of high water, of the substance which the cutwaters now show within that range (see Plate 24), flat pointed arches might be sprung at that level, and the whole superstructure reconstructed in accordance with the prevailing style of the Abbey, Hall, and Palace of Westminster. The longitudinal central groining hereinbefore proposed might well be adopted with excellent effect, lightening the upper works, relieving the thrust of the arches, and greatly economizing the reconstruction, as the old stone would work in well for this purpose, whilst the faces and main vaults were of new. The widening of the water-way by the removal of the springings of the arches out of the water would allow characteristic abutments to occupy the space now taken up by the two first arches of the series of thirteen, as well as the site of the two small land arches, without affecting the current injuriously; and as the flat pointed arch would give much more freedom to the navigation than the semicircular arch affords, independently of the increased lateral space in every bay, the vertical head-way might be taken at an average of that now afforded by the central group. Moreover, the increased space at the approaches obtained by obliterating the useless land arches would allow the accesses to the bridge from the

Elevation of half the length of WESTMINSTER BRIDGE re-modelled to correspond in style with the Houses of Parliament, the arches being of greater span upon the same piers, but reduced in number; the abutments advanced upon the water-way in the places of the obliterated arches, and the road-way lowered to diminish the ascent, whilst the superficial area available for navigation under the arches is not decreased. The dotted bowed lines below indicate the outlines of the present arches, and the dotted lines above the elevation indicate the present elevation of the bridge.





Re-modelled plan of Westminster Bridge at two levels, according to the design in the elevation above it, and, as described in the text on the next page, with approaches lengthened to ease the ascent from, without encroaching upon, the streets leading up to them.

low ground on either side to be greatly improved, and the ascent eased by dividing them to the right and left over the abutments, and so to distribute the rise over a longer space, and afford the means of dividing the going and coming traffic.

This gives occasion to remark further that the approaches to a bridge may be often advantageously disposed in the manner here suggested, to avoid the too common evil of a quick ascent, or the very great expense which carrying constructions far inland may impose, particularly within a town. Such an arrangement is desirable also if it were only for the display of the work itself, as the widened approach and the bending line would bring the elevation of the bridge into view; an advantage which ought to be secured whenever it can be obtained without any unreasonable sacrifice, and more especially when it can be obtained in connexion with a real benefit. Approaches to Westminster Bridge might be thus made to display to the passenger the elevation of the bridge itself, as well as the splendid works within view from the corbelled recesses which in such a case it may not be unwise to provide, and which the design contemplates.

It may be questioned how far the system generally adopted of building towers of masonry to carry the chains of flexible suspension bridges is consistent with sound construction, but there can be little question that as a matter of convenience such constructions are very much in the way; two narrow gorges being established by them in the road-way of a bridge so upborne, through

which foot passengers and carriages must crowd together, or the former must wait, as in the recesses on the old pack-horse bridge, until the stronger and bulkier occupant of the way shall have passed on. Moreover, heavy masses of masonry intercept the graceful lines of the chains, and break the continuity of the floor of the bridge itself, whilst the unsteady character of the suspending and suspended works tend to unsettle the masonry and render it less safe for bearing towers or standards, than components which can be bolted down and together, and rendered certainly secure. The substructures of the bearing towers or standards may be of masonry advantageously, but the superstructure—all above the floor of the bridge, or the road-way—would be much better, both constructively and as a matter of taste, of cast iron. Not, let it be understood, iron columns and entablatures, or imitations of arches in iron, or mock piers, as at Brighton, but simple, plain, and boldly composed standards, well braced and tied together, and yielding as much space to the road-way as the chains themselves yield. The standards and frames of steam engines often take the proposed character, when the machinist has aimed at the end to be answered in the directest manner, without mystifying himself and making his work ridiculous by the imitation of what are considered architectural forms of decoration.

The little suspension bridges lately put up in and about the Regent's Park at London illustrate what is here intended in some degree;—that is, in the use of bearing standards instead of bearing piers or towers,

though the general good effect produced is marred, upon closer observation, by the utter absence of good taste in the details of the compositions. It is not to be understood, therefore, that architecture in its ordinary sense as a decorative art is to be discarded in the composition even of chain suspension bridges; but that the set forms, ordinarily, or, it may be said without impropriety, vulgarly, considered as essential to architectural disposition, will not give true architectural character to any structure, unless they are applied appropriately,—with reference to useful service as well as agreeable effect; and for bridges, as a class of structures, the set forms of architecture are almost wholly inappropriate. Truly, however, architecture does not consist of or in set forms, but in a harmonious disposition of parts with reference to the object to be attained, and in graceful forms and combinations of form in adapting the parts to one another. These are as attainable in the composition of bridges of all kinds,—although the special requisites may vary for every kind,—as they are thought to be in the composition of temples and palaces.

SUPPLEMENT

TO VOL. II.

SPECIFICATION OF CHESTER DEE BRIDGE.

AMERICAN TIMBER BRIDGES.

DESCRIPTION OF THE PLATES.

GENERAL INDEX.

SPECIFICATION FOR A STONE BRIDGE

PROPOSED TO BE BUILT OVER

THE RIVER DEE AT CHESTER,[1]

ACCORDING TO

THE DESIGN OF THOMAS HARRISON, ESQ.,

WITH THE NECESSARY EMBANKMENTS AND ROAD-WAYS THERETO.

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1. THE bridge to be built across the river at the upper end of the Roodee, as nearly at right angles with the stream and as near to the site now marked out as circumstances will permit; to consist of one arch of 200 feet span, and the versed sine to be not less than 40 feet, of two abutments and two dry arches, with their respective abutments and wing walls; and to be constructed according to the plans and sections hereunto annexed, and of the dimensions therein set forth.

2. The water to be kept off by proper dams, and the foundations for the abutments to be sunk within those dams to the several depths figured upon the plans; namely, the abutments on the north side of the river to be sunk to the depth of 21 feet below the springing of the arch, and that on the south side to be sunk 16 feet 4 inches below the same springing: the fronts of the abutments to be of horizontal courses not less than 2 feet thick, and the interior to be laid in radiated courses of strong ashlar in as large blocks as can be conveniently procured, and thus continued over the interior surface of the abutment until they reach the springers of the arch, at which place the arch courses are to be 6 feet in breadth on the bed and gradually to decrease until they reach the key-stone, at which place they are to be 4 feet in

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[1] See wood-cut in page 44 of Mr. Hosking's Preliminary Essay; also the valuable Plates in Vol. I. of the Transactions of the Institution of Civil Engineers.

breadth on the bed. The backing of the arch and of the abutments to be carried up together with the dry arches, the invert arches, and wings, as shown by the longitudinal section marked A No. 1.

3. The foundations for the wings to be sunk as follows: that on the Chester side to be sunk to the depth shown in the drawings, which foundation is to be strengthened by the driving of piles of oak, elm, or beech, or any other suitable timber, to be shod with iron or not as circumstances may require: each pile to be from 12 to 14 inches in diameter, and to be driven with such force as to be able satisfactorily to resist the weight they are intended to carry. These piles to be driven at the distance of from 3 feet 6 inches to 4 feet from centre to centre of each pile; between these piles the earth to be excavated to the depth of about 12 inches, and to be filled with rubble stones pitched close on the edge: the interstices to be filled with good mortar, and small spalls or splinters of stone to be thrust into them, and the whole to be rammed hard down with a heavy hammer within half an inch of the pile heads,—the pile heads having been previously adzed or otherwise levelled.

4. Upon these pile heads and rubble-work the first course of ashlar to be laid, of not less than 18 inches in thickness and of 3 feet in breadth, but as much broader as can be got, and to be laid in headers to tie as far across the foundations in one stone as can conveniently be had. The second course to be not less than 18 inches in thickness and to be a stretching course both front and back of 3 feet in breadth on the bed, to be filled between with rubble backing as large as the cavity will admit. The wing walls on the south side of the river to be sunk to the rock at the supposed depth of 10 feet or thereabouts below the springing of the large arch, the foundations of which to be laid after the manner above described for the foundation of the Chester side.

5. The spandrels and pilasters, the abutments and spandrels of the dry arches, and the wing walls, to be carried up with ashlar not less than 2 feet in breadth on the bed, and not less than 16 inches in thickness, nor less than 18 inches on the joint (observing always that the courses shall diminish in thickness as they ascend). To have bond stones in each

course not more than 12 feet apart, nor less than 2 feet in length on the face, to tie into the wall 4 feet, and to fall nearly over each other in each alternate course.

6. To erect large Doric columns with corresponding entablature on each side the abutments; the columns to be neatly fluted and the capitals moulded; the whole to be done in every respect according to the drawing. The cornice to be 2 feet thick and moulded as shown in the drawings, with a sunk soffit, and to average 4 feet 6 inches on the breadth of the bed.[2] The parapet to be 4 feet high and 15 inches in thickness, having the top slightly rounded to about 1 inch in the centre; the joints to be herring-boned into each other to the depth of three quarters of an inch in the centre of each joint. The arch to be filled up or backed from a line vertical with the face of the abutment, by means of longitudinal spandrel walls of brick, and arched with the same, as shown in the section marked A No. 1, and B No. 2.

7. The road to commence at the Great Gateway of the Castle of Chester, and to fall gradually in a curved line to the pavement where it crosses the City walls, afterwards to rise in the same manner until it comes to the abutment of the bridge. The road on the south side to the extent of about 200 yards will fall from the abutment in the same inclination as on the north side, and will then gradually rise until it forms a junction with the Holyhead Road at Overleigh. The embankments to be 42 feet wide at the top; the slopes to be as three to two; that is, three horizontal to two perpendicular in every instance. The work to commence at the foot of the slopes; the materials to be made to slide towards the centre of the embankments from each side, so that when the work is nearly completed a longitudinal trench in the centre of the road shall remain to be filled up afterwards. The road over the embankments to be 39 feet between the railed fence; to have on each side a foot-path 6 feet wide and in the centre a carriage-road 27 feet wide. To be constructed of a layer of pitched

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[2] This part of the work was subsequently altered to Mr. Harrison's second design of a plain niche with a panel over it, as mentioned in the description of the bridge in Vol. I. of the Transactions of the Institution of Civil Engineers, pp. 207-214.

rubble from the red stone quarry, 8 inches thick; secondly, a course of well-screened gravel, 4 inches thick; and thirdly, a coat of broken sea gravel-stone, 2 inches thick, with a small portion of gravel screenings spread upon the surface to assist the binding. The transverse section of the road to be elliptical, its convexity from side to centre to be 9 inches; the foot-paths to be the same height as the centre of the road-way, to be coated with 3 inches of well-cleaned gravel, covered with a coating of good gravel screenings, 2 inches thick, until it meets the Yorkshire paving at the termination of the parapet wall of the bridge.

8. To have a fence on each side of the road from the Castle to the junction with the Holyhead Road at Overleigh; to be composed of sawed and wrought oak posts 5½ by 6 inches square, and 8 feet asunder, with two heights of sawed wrought red deal rails 3¾ of an inch square, to be fixed arris-wise.

9. The road *over the Bridge* to be made as follows: a stratum 8 inches thick of gravel or refuse of any kind, upon which put a course of pitched rubble, and to be completed as above stated for the approaches. The bridge to be 33 feet within the parapets, and to have a foot-path of Yorkshire stone, 3 inches thick and 4 feet wide, with a curb of the hardest stone from Peckforton Hill, 6 inches wide by 12 inches deep, with proper channels to carry off the water.

10. The stone to be used in the work to be as follows: the foundations and facing of the large abutments and the arch are to be of the best Peckforton Hill stone, with the exception of the springing course, the course under it, the two first courses of arch-stones on each side, the three keying courses, and the quoins of the arch and of the abutments, all of which are to be of Scotch granite, excepting the top moulding of the archivolt, which is to be of Peckforton stone: the Peckforton stone to extend into the abutments and arch as far as shown on the section marked A No. 1, and shaded with light yellow; the remainder of the abutments and backing of the spandrel walls, together with the dry arches, their foundations, spandrels and abutments, and the wings, to be of Chester red stone. The stone to be used in the spandrels of the large arch, the pilasters, columns, cornice, and parapet, to be of Manley, Peckforton, or Kelsale white stone; or stone from any other quarry which

may be considered by the engineer equally suitable for the various parts of the work may be used at the discretion of the contractor. The whole of the stone used to be sound, and set upon its right bed.

MANNER OF EXECUTION.

11. All the beds and joints of the ashlar arch-stones and radiated backing in the abutments to be very straight and truly wrought, gauged, and squared throughout their whole width: the first draft round them to be very straight and true, from which straight cross drafts must be wrought, not more than from 15 to 18 inches asunder; the intermediate spaces may then be axed near and knotted off straight and even with the drafts, in order that they may be set closely and firmly in all directions upon a thin even bed of fine mortar. The faces of all the ashlar up to the springing of the arch to be neatly pitched off all round with a sharp chisel to a straight clean line, and the remainder to be left rough, observing that there be not less than one inch nor more than two inches left upon each face. The soffit of the arch to be plain boasted with an arris rustic of 1½ inch on each bed joint. The external face of the quoins to have an archivolt worked upon them, the top member to be of Peckforton stone, as shown on the transverse section marked A No. 1. The spandrels, columns, pilasters, cornices, and parapets, to be clean boasted.

12. The faces of the ashlar in the land arches, the arch-stones, their spandrels and wings, to have a ¾ of an inch margin run round the face, and the remainder neatly broached off. The whole of the joints of the ashlar and arch-stones throughout the work to be crossed with as nearly the thickness of the stone which it crosses as can possibly be done; if not, as near to the centre of the stone beneath.

The backing of the wings and spandrels to be of gauged and axed courses on the back of their walls, having the edge of their faces neatly pitched off round the stone and on the breadth of bed with binding stones, as before described for the ashlar; the cavity between to be filled with flat bedded rubble stone, as large as such cavity will admit; the whole to be laid in good well-tempered mortar, the remaining spaces to be filled with pouring grout; thin scapplings or spalls are then

to be thrust in amongst it. The backing upon the abutments upon the radiated ashlar to be of flat bedded rubble stone, done in manner as above described.

13. The mortar for the arch and front masonry to be made very fine, and composed of equal parts of lime and clear sharp sand; and that for the interior to be of two parts sand to one of lime; and the whole to be well-tempered. The lime to be the best that can be procured in the neighbourhood.

CONDITIONS.

14. The contractor to find all materials of every description required for the work, except the mine rent for the red stone, the clay for making what bricks may be wanted in the said works, and also the earth for the embankment; each of which must be found by the Bridge Commissioners from the lands belonging to the Corporation of Chester, situate as near to the south end of the intended bridge as possible, without any expense to the contractor, who shall be at the expense of carting the same, and at every other expense connected with or occasioned by the operative execution of the work before stated. The whole of the work, both as regards the quality of the materials, the execution and mode thereof, to be done to the entire satisfaction of the engineer to the commissioners, who shall be at liberty to direct the manner of distributing the various materials to be used in the permanent structure of the work, according to his own discretion, as well as to make any such alterations, additions, or deductions to or from the plan, in the progress of the work, as he shall see necessary; but should he so dispose of such materials as to require a larger quantity of any sort or sorts than what is shown or stated in the plans and sections, such additional quantity shall be allowed to the contractor, either in value or difference, at a fair valuation; but if other materials be substituted by him, such alterations to be valued so as to give or take from the quantity estimated, or the value thereof, as the case may be; the estimate being made from the plans marked from A No. 1 to G No. 7; and should any alteration be made in any of the works, whatever they may be, the same shall in nowise invalidate or make void the contract, but to be measured and

valued, and either added or deducted as the case may require. The work to be commenced upon forthwith, and the whole to be completed according to the plans and this specification within four years.

Should it at any time appear to the surveyor during the execution of the work that the contractor is neglecting or doing any part of the work contrary to the true spirit of the plans and this specification, the committee shall have it in their power, after giving twenty-eight days' notice in writing to the contractor, either personally or by leaving the same at his usual place of abode, to take the work out of his hands and employ others to finish the works, provided he does not proceed, immediately after such notice has been given, to complete the works according to this specification, and to the satisfaction of the engineer; and whatever money may be due to the contractor at the time being shall remain in their hands until the bridge is completed; and all loss or damage that may be sustained through the misconduct or neglect of the contractor to be defrayed out of it, so far as the same will extend, and the remainder of such loss or damage to be defrayed by the contractor out of his own proper monies.

The instalments for the bridge are to be made to the contractor as follows, viz.: the value of the whole of the work contained in the first abutment to be paid for in full so soon as such abutment is carried up to the springing of the arch; the second instalment to take place when the second abutment has in like manner been carried up to the springing of the arch, and to be paid for in full; the third instalment also to be paid in full when the arch is turned and the centre has been taken out; the fourth instalment to be paid in full when the masonry has been carried up to the height of the road; and the last instalment when the whole of the work has been satisfactorily completed; observing that the commissioners shall be at liberty to make an advance to the contractor between each instalment, should he require it, by a per-centage on the work done and upon the materials of all descriptions upon the ground, according to the valuation thereof by the surveyor appointed by the commissioners: but between the second and third instalments, the commissioners shall be

bound to allow the contractor, according to such valuation, a per-centage of not less than 20 per cent.; but they may make it as much more as they may feel disposed to advance. The whole of the several instalments to be calculated upon the engineer's valuation of the work done and the prices of the materials, and according to the detailed prices of the whole work, to be lodged with the clerk to the commissioners. The materials and machinery of all descriptions brought or laid upon, at, or near the work, to be considered as the property of the commissioners, and not to be removed away without their consent. In respect of the embankments, the first instalment of one-third of the estimate of £7500 to be paid when the engineer shall have certified that one-third of the earth-work has been completed; the second instalment of another third of the said estimate to be paid when the engineer shall have certified that another third of the earth-work shall have been completed; and the remainder of the £7500 to be paid when the engineer shall have certified that the whole embankment and the road over the same, together with its fences, have been completed according to the specification,—the commissioners having liberty, if they shall think fit, to allow such sum between the second and third instalments, on account of such third instalment, as the engineer shall recommend: the estimate of the sum agreed to be paid for the foregoing work, as herein specified, being £29,300 for the bridge, and £7500 for the embankments, being together £36,800.

(Signed) James Trubshaw.

And I further agree to bind myself in the sum of £10,000 for the due performance of the work.

(Signed) James Trubshaw.

(Signed) Jesse Hartley, Engineer.
Chester, 25th January, 1827.

PRACTICAL DESCRIPTION

OF

THE TIMBER BRIDGES, &c. ON THE UTICA AND SYRACUSE RAILROAD,

IN THE UNITED STATES.

BY B. F. ISHERWOOD, C.E., NEW YORK.

As all the bridges constructed on the Utica and Syracuse Railroad resemble each other in the framing of the timber, in the manner of scarfing, (which is technically termed fishing,) and in the connexion of the floor beams, it is unnecessary to introduce geometrical plans, elevations, and sections in every Plate: accordingly they are confined to the delineation of the 84 feet span, of the 88 feet span, and of the 100 feet span bridges.[1]

For the same reasons, specifications have not been appended to every plan; as in general they would be but mere repetitions of each other. They are given for the heavy bridging across the Oneida Creek and Valley, for the 100 feet span bridge over the Mohawk River, and for the trestle bridge over the Onondaga Creek and Valley; and these are sufficiently numerous to explain

[1] Engineers of the works, O. H. Lee, Esq., and Major C. B. Stuart.

the manner of construction and the quality of material used.

The trusses for the other spans are represented in isometrical perspective, giving in one view the manner of framing, connexion of timbers, and all the detail, as contrived and executed in the original: appended to each Plate is a short description, stating the principal dimensions, and from the attached estimates of timber may be drawn their various sizes. Thus the practitioner, if unacquainted with the principles of the style of drafting (where isometrical projections are alone used), is enabled to recover the plan in geometrical proportions.

The trestle bridge over the Onondaga Creek is not upon the line of the Utica and Syracuse Railroad, but was constructed by the Company at its western terminus, for the Auburn and Syracuse Railroad, which joins it there, and thence extends westward.

List of Prices for which the Bridges on the Utica and Syracuse Railroad were constructed.

Location and description of Bridges.						Foundation.		
Name of Bridge.	No. of feet span.	No. of spans.	Kind of foundation.	Price ℔ lineal ft. for superstructure, finished according to plans.	Masonry in piers and abutments per cubic yard.	Excavation per cubic yard.	White pine timber ℔ M. ft. board measure.	Oak piles driven ℔ foot.
				$	$	$	$	$
Mohawk River, east of Rome . .	100	1	Pile bents	17·66				0·75
Erie Canal, near Canastota . .	88	1	Stone abut^s.	18·00	8·00	0·18	10·00	
Oneida Creek, near Oneida Castle	84	1	" "	19·00	4·75	0·45	15·00	
Mohawk River, at Rome . . .	60	2	" "	12·88	5·00	0·18	9·00	
Erie Canal, at Rome	60	1	Pile bents	10·50		0·25	10·00	0·40
Butternut Creek	40	2	" "	8·13				0·35
Limestone Creek	40	2	" "	8·13				0·35
Chittenango Creek	40	2	" "	8·13				0·35
Canaseraga Creek	40	1	" "	8·13				0·35
Canastota Creek	40	1	" "	8·13				0·35
Coweslon Creek	40	2	" "	8·13				0·35
Oriskany Creek	40	2	" "	8·13				0·40
Sanquois Creek	40	2	" "	8·13				0·40
Lake Brook	30	1	" "	8·65				0·40
Canaseraga Raceway	30	1	" "	8·65				0·40
Oneida Valley	30	60	Piles & bents	5·00	4·93	0·20	10.00	0·24
Alexander's Brook	30	1	Timber bents	8·65				0·40
Brandy Brook	30	1	" "	8·65				0·40
Stony Brook	30	1	" "	8·65				0·40
Mud Creek	30	2	Pile bents	8·65				0·40
Spring Brook	30	1	" "	8·65				0·40
Onondaga Creek	30	20	Piles	5·08				0·35

Remarks:—The above prices include the iron and all materials necessary to finish the superstructure of bridges according to plans and specifications. The masonry to be laid in water lime.

GENERAL DESCRIPTION.

The bridges constructed on the Utica and Syracuse Railroad are of the kind denominated plank bridges. They are formed of thin sawed timber, of equal thickness, disposed in a system of triangular bracing, so as to throw the entire strain upon the abutments and piers. From the disposition of the grade line, which is generally but a few feet above the surface of high water, there remained no room to place the road-way upon the top of the framing: this occasioned the necessity for suspending, by vertical posts, a stringer to carry the floor beams that support the rails, and also to receive the ends of the braces and arches, thus bringing their lateral thrust into a longitudinal strain upon it. Each stretch or span of the bridge is composed (according as the road-bed was graded for one or two tracks) of two or three vertical truss frames, formed of white pine timber, arranged in the manner represented in the plans. These frames simply rested upon the foundations, over which the stringers were notched. The timbers with which the frames are composed were brought into close contact in the desired position, and there securely bolted and strapped together. The floor timbers were placed at right angles to the

sides of the frames, and were generally notched down and laid without any fastening upon the upper sides of the stringers. In some cases, however, where the grade line approached the surface of the water very nearly, they were suspended beneath the stringers, by stirrup irons, as shown in Plate 61, fig. 2. The framing of the diagonal braces between the posts, and also of the braces and arches that abut into the stringers, was effected by cutting completely through the posts and stringers, forming a square step at the ends of the braces and arches to fit the notch, and, simply bringing them in contact without the use of the mortise and tenon joint, fasten them securely to an adjacent post. In the absence of a post at the ends of the stringers, the principal abutting braces with the arches were strapped down upon the stringers. The posts are single, and the stringers, arches, and braces were fastened in pairs upon their sides by screw bolts and trenails. Whenever scarfing was necessary it was effected by bringing the square ends of the timbers together, and bolting iron bars along their sides, having first inserted a block of wood, of the thickness of the posts, between the opposite timbers, at the place of scarfing. As the truss frames are very thin, in proportion to the superficies of their sides, collateral security was necessary, to prevent them from warping, or falling from their vertical position: this was effected by gallows frames, formed by running up opposite posts in the opposite frames, and connecting them crosswise by a cap piece, at a sufficient height to permit the passage of the trains beneath: to stiffen the

frames, braces were then inserted in the angles. Directly at the sides of the posts that formed the gallows frames were laid floor beams, of the necessary length to project 5 or 6 feet beyond the sides of the trusses: from their projecting ends braces were inclined against the truss, and fastened so as to bring them nearly in a line with the gallows frame braces. Each truss frame is protected from the action of the weather by a casing of white pine inch boards, placed vertically upon its sides and ends, and matched at the edges with a tongue and groove, so as to entirely exclude the water. Between the casing and the timber of the frame, fuming pieces were interposed, 1 inch thick, to secure a free circulation of air from beneath, the bottom remaining uncovered. The casing was thoroughly nailed and secured to the frame. Each truss frame is covered by a coping plank that projects over its sides: this coping plank was bevelled on its upper surface, so as to carry off the water. Under each projecting edge was made a groove, and a moulding inserted in the angle effectually prevented the access of water. The truss frames are separated by a distance of 11 feet. The flooring of the bridge is composed of pine or hemlock plank, well spiked or pinned to the floor beams. The ends of the plank are separated by a slight space from the sides of the truss frames. All the timber used in construction was of the first quality of square-edged white pine timber, perfectly sound, and free from black or loose knots, windshakes, wormholes, and sap, and was framed and braced in the most accurate and workmanlike manner, so as to secure

the whole strength of the timber. Each shoulder and joint was made to fit and bear with the utmost precision. The side and end coverings of the truss frames were thoroughly covered with two coats of oil and white lead, as were also the coping, gallows frames, and outside braces.

The bolts and straps used were of the best quality of American wrought iron, of the specified sizes. The screw bolts were of round inch iron (driven through $\frac{7}{8}$-inch auger holes), with substantial square heads, nuts, and washers: the screws were 2 inches long, with a clear and strong thread. As the lateral thrust of the braces and arches that compose the truss frames is resolved into a longitudinal strain upon the stringers that receive them, the abutments and piers are consequently subjected only to the downward pressure of the weight of the bridge. The engineers were thus enabled to make the foundations of less dimensions, and of less durable materials, than would have been required for a bridge with arches thrusting directly upon them. For this purpose pile bents were adopted, as the cheapest and most efficacious means to insure the object. The piles were of straight and sound white oak timber, 12 inches square, and free from wane and loose or black knots. In all cases they were driven sufficiently deep to reach the hard or solid bottom. They were sawed off at the proper level to receive the cap pieces, which were fastened on by the mortise and tenon joint, and secured by trenails, 2 inches in diameter, well wedged at their heads.

Each bent consists of two rows of piles, driven in straight lines, on a line corresponding with the thread of the stream. Each row consists of four piles for single track, and seven piles for double track bridges. These rows were placed at distances of 1, 2, or 3 feet apart, varying with the spans of the bridges, and were connected together by cross caps, dovetailed into the longitudinal caps.

The piles were so arranged in the rows that one was situated immediately beneath each truss. The cap pieces are of the same quality with the piles, and were framed and finished in a substantial and workmanlike manner.

The sides of the bents were planked up from 1 foot below low-water mark to their tops with sound 3-inch white oak plank, laid edge to edge, and strongly pinned to the piles. The larger bents were generally filled with cobble stone.

The trestle bridges were used in crossing wide valleys, considerably depressed below the grade line: they were a cheap substitute for embankments, and effected a great saving of time in construction. As these decay, in the progress of years, they are filled up with earth, hauled by locomotive engines at a reduced cost. These structures are formed by erecting, at short distances, wooden piers, composed of single bents: across them horizontal stringers were laid, and supported at equal spaces by braces abutting in the posts of the bents. A railing was erected upon the sides, and the stringers were covered with plank, upon which the railway was

constructed. All the timber used in their construction was of the same kind and quality as that used in the construction of the truss bridges, and the workmanship was the best description of carpentry. The trenails which hold the tenons in their mortises are octagonal in their section, and were driven through auger holes of a less diameter: their heads were spread by wedges after they were driven home.

The bents were placed on piles, driven to a hard bottom, and sawed off at the level of the ground, and also on foundation walls of stone masonry, according as the nature of the soil would permit.

Plate 61.—Fig. 1 is an isometrical projection of one-half of a bridge of 40 feet span. The truss, one of the simplest that can be contrived, is very cheap, and easy of construction. It consists of two stringers at bottom, and, at a distance of $5\frac{1}{2}$ feet between them, of two horizontal abutting pieces at top, with sloping abutting braces footing in the stringers: vertical posts, placed $6\frac{2}{3}$ feet from centre to centre, connect these, and project 6 inches above and below the truss.

Upon each side of the posts are a pair of minor braces, footing in the stringers, and abutting at the centre of the span beneath the horizontal abutting piece.

Bolting blocks are inserted between the ends of the stringers, and between the lower ends of the abutting braces, forming a solid mass to bolt through. The timbers are fastened together by wrought iron screw bolts,

and the sloping abutting braces are tied down upon the stringers by wrought iron bands. The flooring beams are placed equidistant 5 feet from centre to centre, notched down 1 inch, and laid upon the stringers: two of them are produced $5\frac{1}{2}$ feet beyond the exterior face of the truss, and from their extremities braces are inclined and spiked against the horizontal abutting piece, thus giving the truss a greater base, and assisting to sustain it in its vertical position upon the abutments.

A railing is placed at each end of the truss, which is then boarded up, coped, and painted.

Across the flooring beams are laid the wooden rails, 8 inches square, and upon them are spiked the oak ribbons and iron plate that form the track. The intermediate spaces of the floor are laid with 3-inch thick pine plank.

The abutments are formed of two rows of piles: the piles, 12 inches square, are placed 2 feet from centre to centre transversely, and 4 feet from centre to centre longitudinally, united at top by a longitudinal cap, 12 inches square, with a cross cap beneath each truss: the exterior of the abutments is then cased up with 3 inch thick oak plank.

Estimate of Timber for a single Track Bridge of* 40 *feet Span,* 11 *feet in the clear between trusses.

No. of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
4	Stringers	White pine	46	4 × 15	920
4	Horizontal abutting pieces	" "	14 1/12	4 × 15	281 2/3
8	Sloping " braces	" "	17 1/4	4 × 15	690
8	Minor " "	" "	9 1/2	4 × 9	228
4	Railing posts	" "	4 1/4	6 × 4	34
4	" caps	" "	10 1/2	3 × 12	126
4	Coping plank, not shown in Plate	" "	10 1/2	3 × 16	168
2	" " "	" "	14 1/2	3 × 16	116
4	" " "	" "	6	3 × 16	96
4	Bolting blocks	" "	4	4 × 15	80
4	" "	" "	2	4 × 15	40
6	Flooring beams	" "	13 2/12	7 × 12	553
2	" "	" "	24 1/3	7 × 12	340 2/3
4	" " braces	" "	8	6 × 6	96
	Flooring plank, not shown in Plate	" "	15	3 × 12	1334
	Siding boards " "	" "	9	1 × 4 to 10	915
	" " " "	" "	6	1 × 4 to 10	495
6	Main posts	" "	9	4 × 15	270
4	" "	" "	5 3/4	4 × 15	115
					6898 1/3

Estimate of Iron.

68 bolts of 1 inch round wrought iron, 12 inches long in clear of head and nut.

heads and nuts for ditto, 2 inches square by 1 inch thick.

washers for ditto, 3 inches diameter by 1/4 inch thick.

4 wrought iron bands, 2 feet 8 inches by 12 inches in clear, of 2 5/8 inches by 3/8 inch thick iron.

Total weight of iron, 568 lbs.

Fig. 2 is an isometrical projection of one-half of a bridge of 30 feet span: the truss is similar to fig. 1. In a bridge of this span the stringers and horizontal abutting pieces are separated by a distance of 3 1/2 feet between them. The posts are placed 10 feet from centre to centre. Bolting blocks are inserted between the ends of stringers and between the lower ends of sloping abutting braces, as in fig. 1.

The flooring beams are suspended beneath the truss by stirrup irons that pass between the stringers, and over oak wedges, laid upon the stringers in contact with the sides of the posts. At right angles to the flooring beams, and notched three inches over them, are laid heavy longitudinal runners, of the necessary depth to make their upper surfaces in the same plane with the top of the stringers. The whole width of the bridge is then floored over with pine plank, and the 8 inches square rails are laid directly over the longitudinal runners, with the oak ribbons and iron plate for the track, as in fig. 1. From the ends of the floor timbers, braces incline to the truss, as in fig. 1; and the bridge is finished in all respects the same, with the omission of the railing, and the substitution of oak trenails for iron bolts.

The foundations are of single timber bents, composed of mud sill and cap united by vertical posts: braces are introduced at the ends, on the same inclination with the embankment behind them. The rear is then planked up.

The sills, cap pieces, and vertical posts, are of 12 inches square pine timber.

The braces are of 6×9 inches pine scantling.

The planks are of 3 inches thick pine.

Estimate of Timber for a single Track Bridge of 30 *feet Span,* 11 *feet in clear between trusses.*

No. of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
4	Stringers	White pine	34	4 × 15	680
4	Horizontal abutting pieces	„ „	10¾	„	215
8	Sloping „ braces	„ „	13	„	520
2	Coping planks, not shown in Plate	„ „	12¼	3 × 16	98
4	„ „ „	„ „	13	„	208
8	Bolting blocks	„ „	2½	4 × 15	100
2	Flooring beams	„ „	24	10 × 12	576
4	„ „ braces	„ „	6¾	6 × 6	81
2	Longitudinal runners	„ „	34	8 × 15	680
4	Posts	„ „	7	4 × 15	140
4	Wedges for stirrup irons	„ oak	1	6 × 10	20
	Flooring planks	„ pine	13⅛	3 × 12	1185
	Siding boards, not shown in Plate	„ „	6⅛	1 × 4 to 10	572
					5075
40	Trenails of white oak, 1¼ inch diameter, 8 square				
8	Trenails of white oak, 1 inch diameter, 8 square				

Estimate of Iron.

4 bands, 2 feet 3 inches by 12 inches in clear, of 2⅝ inches by ⅝ inch wrought iron.

4 stirrup irons, 33 inches by 10 inches in clear, of 2⅝ inches by ⅝ inch wrought iron.

Total weight of iron, 280 lbs.

Plate 62 is an isometrical projection of a truss for a bridge of 60 feet in span between abutments: it is, as may be seen, an enlargement of the 40 feet span in Plate 61, fig. 1. The additional strength and stiffness required for the additional length is obtained by the insertion of a pair of arches and counter braces.

As the depth of the truss increases in proportion with the span of the bridge, while its thickness continues only the same, it obviously requires more security against the tendency of its enlarged superficies to warp and change its vertical position: this is effected by means of

gallows frames, formed by running up posts in opposite trusses, and connecting them at top by caps: the frame is then stiffened by inserting braces in the angles. The depth of this truss, measuring from the bottom of the stringer or chord to the top of the horizontal abutting piece, is 10 feet 3 inches. The posts are placed equidistant 7 feet 9 inches from centre to centre.

This bridge rests on a similar foundation, and is in all respects finished in the same manner as the 40 feet span in Plate 61, fig. 1.

Estimate of Timber for a double Track Bridge of* 60 *feet Span,* (3 *trusses,*) 11 *feet in clear between trusses.

No. of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
6	Stringers or chords	White pine	66	4 × 15	1980
6	Horizontal abutting pieces	" "	31½	4 × 12	756
12	Sloping " braces	" "	19¾	"	948
12	Minor " "	" "	18½	"	888
12	Diagonal braces between posts	" "	11¼	4 × 8	360
6	" " "	" "	8¼	"	132
2	Gallows frame caps	" "	25⅓	8 × 8	270⅓
8	" " braces	" "	6	6 × 8	192
6	Railing posts	" "	6½	4 × 6	78
6	" caps	" "	10¾	3 × 12	193½
18	Arch pieces	" "	16	4 × 12	1152
12	" "	" "	7½	"	360
13	Floor timbers, not shown in Plate	" "	25⅓	7 × 12	2305⅓
2	" " "	" "	37½	"	525
4	" braces "	" "	9¾	7 × 9	204¾
6	Coping plank, not shown in Plate	" "	16½	3 × 16	396
6	" " "	" "	8½	"	204
6	" " "	" "	11	"	264
9	Main posts	" "	11¼	4 × 12	405
6	" "	" "	7	"	168
6	Gallows frame posts	" "	18½	"	444
6	Bolting blocks	" "	4¼	"	102
6	" "	" "	5½	4 × 15	165
6	" "	" "	2¼	4 × 12	54
	Flooring plank, not shown in Plate	" "	15	2½ × 12	3630
	Siding boards " "	" "	11¼	1 × 4 to 10	3004
	" " " "	" "	7	"	903
27	White oak trenails, 1¼ inch diameter, 8 square, and 12 long				20,083⅞
30	White oak trenails, 1 inch diameter, 8 square, and 8 long				

Estimate of Iron.

195 bolts of 1 inch round wrought iron, 12 inches long in clear of head and nut.

heads and nuts, 2 inches square by 1 inch thick wrought iron.

washers for ditto, 3 inches diameter by $\frac{1}{4}$ inch thick wrought iron.

6 bands, 3 feet 3 inches by 12 inches in clear, of $2\frac{5}{8}$ by $\frac{5}{8}$ inch wrought iron.

6 bands, 2 feet 6 inches by 12 inches in clear, of $2\frac{5}{8}$ by $\frac{5}{8}$ inch wrought iron.

Total weight of iron, 1711 lbs.

Plate 63 and 63* show a considerable alteration in the system of trussing. The truss here represented of 88 feet span is much simpler in its design than that of 82 feet span, shown in Plate 64*: like that, this possesses the abutting pieces, abutting piece braces, and stringers, united by vertical posts, stiffened by counter braces; but, substituted for the arches, minor abutting pieces, and their sloping braces, are a succession of parallel braces, notched into the stringers opposite to a post, crossing an intermediate post, and notching beneath the abutting piece opposite to a third post: they are bolted to the posts at their intersections, and secured by trenails to the counter braces.

Upon the exterior of this truss is bolted a pair of arches (parallel) that thrust against the stone abutments: they descend below the stringers, and are fastened to the end posts produced downwards: they form a great addition of strength and beauty.

In casing up the bridge the arches are cased separately, adorned with mouldings, and relieved by a darker tint.

In this bridge the floor timber braces were retained, but the gallows frames were omitted, as it was deemed that sufficient security against lateral twisting was obtained by the extra width of truss given by the arches.

A railing was placed at the ends of the truss, and the details in all respects were the same as described for the other bridges.

The depth of truss, from under side of stringer to upper side of abutting piece, is $10\frac{7}{12}$ feet.

The distance from centre to centre of posts is 8·70 feet.

The span of the arch is 86 feet, and the rise to its intrados is $13\frac{1}{2}$ feet.

The foundations were stone abutments, with wings flared, executed in a good style of workmanship, but possessed of nothing in the design to warrant particular notice.

Estimate of Timber for a double Track Bridge of 88 *feet Span,*
(3 *trusses,*) 11 *feet in clear between trusses.*

No. of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
6	Chords or stringers	White pine	63	4 × 16	2016
6	,, ,,	,, ,,	37	,,	1184
6	Horizontal abutting pieces	,, ,,	52¾	4 × 15	1582½
12	Sloping ,, braces	,, ,,	25¼	,,	1515
36	Truss braces	,, ,,	21½	4 × 12	3096
12	,, ,,	,, ,,	11	,,	528
21	Posts	,, ,,	11½	,,	966
6	,,	,, ,,	8½	,,	204
6	,, at ends	,, ,,	16¼	,,	390
6	,, for railing	,, ,,	6½	4 × 9	117
6	Railing pieces	,, ,,	14¾	4 × 12	354
18	Diagonal braces between posts	,, ,,	12½	,,	900
6	,, ,, ,,	,, ,,	10½	,,	252
6	,, ,, ,,	,, ,,	6½	,,	156
12	Arch pieces	,, ,,	21	4 × 18	1512
18	,, ,,	,, ,,	18¾	,,	2025
17	Floor timbers	,, ,,	26¾	7 × 12	3183¼
3	,, ,,	,, ,,	41	,,	861
2	,, ,,	,, ,,	11$\frac{10}{12}$	,,	165⅔
6	,, ,, braces	,, ,,	11¾	7 × 9	370⅛
6	Coping plank	,, ,,	27	2½ × 16	540
6	,, ,,	,, ,,	10	,,	200
6	,, ,,	,, ,,	15	,,	300
6	Bolting blocks between stringers	,, ,,	4	4 × 16	128
6	Splicing ,, ,,	,, ,,	7¾	,,	248
6	Bolting ,, abutting braces	,, ,,	4¾	4 × 15	142½
12	,, ,, feet of arches	,, ,,	2	4 × 12	96
	Siding boards	,, ,,	11½	1 × 4 to 10	6900
	Flooring plank	,, ,,	15 & 20	2½ × 12	5500
					35,432$\frac{1}{24}$
36	Trenails of white oak, 1½ inch diam., 8 square, and 12 long				
12	Trenails of white oak, 1 inch diam., 8 square, and 7 long				

Estimate of Iron.

192 bolts of 1 inch round wrought iron, 12 inches long in clear of head and nut.

heads and nuts for ditto, 2 inches square by 1 inch thick.

washers for ditto, 3 inches diameter by ½ inch thick.

108 bolts for splicing bars of 1 inch round wrought iron, 12¾ inches long in clear of head and nut.

heads and nuts for ditto, 2 inches square by 1 inch thick.

washers for ditto, 3 inches diameter by ½ inch thick.

18 splicing bars of wrought iron, $10\frac{1}{2}$ feet long, 3 inches by $\frac{3}{4}$ inch thick.

6 wrought iron bands, 3 feet by 1 foot in clear, of $2\frac{5}{8}$ by $\frac{5}{8}$ inch thick iron.

93 bolts for arches, of 1 inch round wrought iron, 20 inches long in clear of head and nut.

heads and nuts, 2 inches square by 1 inch thick.

washers for ditto, 3 inches diameter by $\frac{1}{4}$ inch thick.

Total weight of iron, 4328 lbs.

BRIDGING OVER THE ONEIDA CREEK AND VALLEY.

The crossing of the Oneida Creek Valley constitutes the heaviest piece of bridging on the line of the Utica and Syracuse Railroad.

The ground is here depressed from 15 to 30 feet below the level of the surrounding table-land: as the approaches furnished no excavation for embankment, trestle-work was adopted as a cheaper substitute.

The valley at this point is about 2000 feet broad, and a trestled structure is carried over it in spans of 30 feet, measuring from centre to centre of bents.

The creek is crossed by a span of 84 feet, which will be hereafter described. Referring to Plate 66, it will be perceived that the trestle-work is of the most economical character; the stringers rest upon single bents, composed of sill and cap piece, with vertical posts between them, stiffened by cross braces of plank pinned against them.

The upper surfaces of the stringers are united by a continuous flooring of plank, fastened by spikes. At

distances of 10 feet the stringers are supported by braces, which abut in the posts, and are held in position by the mortise and tenon joint.

The whole is surmounted by a plain railing, framed upon the exterior by notching out half the lower ends of the posts, then setting them upon the stringer, and spiking their descending parts against its side. Around the posts is placed a projecting coping plank, which covers the ends of the flooring planks, and presents a fair appearance to the eye.

The railing, coping plank, and side of the stringer, are painted; and, viewed from above, the structure presents the appearance of a level platform.

The width is 23 feet and 7 inches in the clear between the railing; a double track is laid throughout, formed by cross ties, of the same size and distance apart as are used in the superstructure for the road-bed; across these, and directly above the stringers placed beneath, are laid the rails and iron plate.

The bents for the trestle-work east of the Oneida Creek rest upon foundation walls of stone masonry; those on the west rest upon a foundation of bearing piles.

The span of 84 feet, (see Plates 64 and 64[a] of span of 82 feet, isometrically drawn,) it may be seen, shows a gradual increase of strength over the 60 feet span; an extra set of abutting pieces and their attendant braces are added, and the timbers are increased in section throughout.

This span is the most perfect adaptation of the prin-

ciple of trussing used on this railroad; the bridge is exceedingly rigid, and bears the heaviest loads without being sensibly affected.

The timbers are secured in position by bolts alone.

In this truss the distance between the stringer and abutting piece is 10¼ feet; the posts are placed 9 feet from centre to centre, and the counter braces are notched into them. The gallows frames were constructed in the manner of those in the 60 feet span bridge; and all the detail and finishing were of the same character described for that span.

The abutments were partially composed of rectangular masses of masonry, executed as per specifications for a height of 10 feet above low water, and were raised to the level of the stringers by wooden frames: the stringers of the trestle-work join upon them, which accounts for their pier-like appearance.

Specification of a Bridge to be constructed for the Utica and Syracuse Railroad Company, across the Oneida Creek and Valley, reference being had to the plans of the same, in the office of the Engineer of said Company.

The bridge is to be composed of 1 span (truss) of 84 feet, over the Oneida Creek; 20 spans (trestle) of 29 feet each, (see Plate 66,) east of, and 40 spans (trestle) of 29 feet each, west of said 84 feet span, and extending across the Oneida Creek Valley. If in the opinion of the engineer it may be deemed necessary or advisable to add to, or deduct from, the length of said bridge, it is

hereby understood that the addition or deduction shall not vary more than five 29 feet spans, at each or either end, from the number of spans specified above.

Foundations for the 84 *feet Span.*

A pit, 38 feet long and 15 feet wide on the bottom, shall be excavated to a depth of 3 feet below the bed of the creek.

The bottom of said pit to be levelled off, and two courses of 6 inches thick timber to be laid crosswise, projecting 2 feet from the base of the masonry on every side.

Masonry.—The masonry shall commence at the level of 16 feet below the stringer of the bridge; at which place the abutment shall be a parallelogram 30 feet long and 7 feet broad; each face shall be laid with a batter of 1 in 12 for a vertical distance of 12 feet, where an offset of 6 inches shall be made, and the masonry continued on the same batter for a further distance of 4 feet.

The masonry to be composed of good sound building stone, well quarried, with flat beds and square joints; no stone to be used of less size than $6 \times 12 \times 18$ inches.

The corner stones to be well bonded into the body of the work, and none to be of a size under $12 \times 18 \times 30$ inches, with their beds and sides squarely dressed and truly set.

The face stones to be laid on their broadest beds, and disposed in alternate layers of headers and stretchers; the headers to compose one-third part of the masonry, and to extend through at least two-thirds of its thickness.

The face stones to have their beds and ends dressed square, and to be laid with a full bearing surface. The face of the abutments to be hammer-dressed, ranged in courses and breaking joints. When finished, every joint to be neatly pointed.

Carpentry.—The abutment from the top of the masonry shall be carried up to the level of the bridge stringers, by a trestle-work of wood constructed as follows:

Two parallel vertical frames shall be erected, at a distance of 3 feet asunder; they shall be strongly braced, and united at top and bottom by cross ties.

Each frame shall consist of a sill and cap piece, separated by a distance of 14 feet. The sill and cap piece of each frame shall be united by seven vertical posts, tenoned into them and well trenailed. Upon the outside of the posts, braces shall be pinned, as shown in plan. The cross ties which unite the frames at top and bottom shall be dovetailed 4 inches into the sills and cap pieces.

The posts shall be so disposed that one pair shall be immediately beneath a bridge truss. The three pairs directly under the bridge trusses shall be stiffened by cross braces, pinned against either side, as shown in plan.

The trestle shall be made in a workmanlike and substantial manner; every joint is to fit and bear with the utmost precision. All the trenails used are to have their heads well wedged; to be 1 inch in diameter, and octagonal in section.

The timber to be of sound white pine, free from sap,

windshakes, wormholes, or large loose knots, and to be of the sizes specified in the subjoined estimate of timber.

Estimate of Timber for one Trestle Frame, forming part of an Abutment for a Bridge of 82 *feet Span, over Oneida Creek.* —*See Plate* 65.

No. of pieces.	For what purpose used.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
2	Cap pieces	White pine	28	12 × 12	672
2	Sills	„ „	28	„	672
14	Posts	„ „	15	„	2520
10	Cross ties	„ „	5	„	600
4	Side braces	„ „	9	3 × 9	81
4	„ „	„ „	15½	„	139½
4	„ „	„ „	21½	„	193½
4	„ „	„ „	15	„	135
4	„ „	„ „	8½	„	76½
18	End „	„ „	8	„	324
					5413½
116	Trenails of oak, 1 in. diar. & 12 in. long				
244	„ „ „ 15 „				
138	„ „ „ 18 „				

Estimate of Cost for one Abutment.

6525 feet, board measure, white pine plank in foundation . .	@ $15·00 ℘ M. B. M. .	$97·87½
5413½ feet, board measure, white pine plank in trestle frame .	@ $15·00 „ „ . .	,81·20
Workmanship on ditto, including cost of trenails		170·00
161·71 cubic yards stone masonry	@ $4·75 ℘ cubic yard .	768·12
		$1117·19½

Note.—The above estimate is founded on the prices contracted for, and shows the actual cost of the abutment.

Superstructure for the 84 *feet Span.*—*See Plate* 64.

The superstructure of the bridge is to be composed

of three parallel vertical truss frames, placed upon the foundations at right angles, and separated from each other by distances of 11 feet.

The truss frames are to be composed in the manner shown on the plan, and the timbers to be firmly bolted together.

All the timber in the superstructure shall be good, sound, well-seasoned, square-edged white pine, free from sap,—that has been sawed at least six months, and is of the dimensions marked on said plan, and stated in the annexed estimate of timber.

The whole shall be framed and braced, as shown on said plan, in the most substantial and workmanlike manner: each shoulder and joint must fit and bear with the utmost precision: every brace and bearing timber that has not a perfect fit and bearing will be rejected. The flooring beams are to be placed at right angles to the truss frames. The joints of the arch pieces to be in the line of radii to the arc. The string timbers to be spliced as shown on the plan, with three splicing bars to each joint: said splicing bars to be 11 feet long, pierced with holes and fitted with spurs on the ends: one hole in each end of every bar is to be so placed that the bolt will pass through the adjacent post.

A splicing timber 5×18 inches, and of such length as will fill the space between the posts, shall be inserted between the stringers, and the bolts shall be screwed until all the timbers are forced into close contact.

The floor is to be formed with sound $2\frac{1}{2}$ inches thick

white pine plank, laid edge to edge, and firmly spiked to the floor beams. The sides of the truss frames are to be cased up with sound well-seasoned white pine tully boards, planed, matched, and nailed on vertically edge to edge in close contact. Fuming pieces are to be interposed between the casing and truss frame.

The trusses to be coped with sound white pine plank, projecting 2 inches over their sides: the upper surfaces of said plank to be properly bevelled for throwing off the water, and the under surfaces to be wrought with a groove.

Painting.—The siding of the trusses and the gallows frames to be covered with two coats of oil, of such colour as may be directed.

Iron.—The bolts to be of the best round wrought iron, 1 inch in diameter, to be driven through $\frac{7}{8}$-inch auger holes; the nuts and screws to be cut with a perfect thread. The heads of the bolts to be firmly fastened on; to be of $\frac{3}{4}$-inch thick iron by 2 inches square. The nuts to be of the same dimensions as the heads, and provided with washers.

In such places as the engineer shall permit, white oak trenails may be used.

The bands to be of the best bar iron, $2\frac{1}{2}$ by $\frac{5}{8}$ inch, firmly welded together.

The splicing bars to be of $3 \times \frac{3}{4}$ inch thick iron, of the same quality with the bands.

Foundations for the 29 *feet Spans.—See Plate* 66.

For the spans east of the 84 feet span, the foundations

shall be prepared by digging a trench 3 feet below the surface of the ground, and of sufficient length and breadth to receive the masonry. A double course of $2\frac{1}{2}$ inches thick pine planks to be laid crosswise, and a wall composed of good sized building stone, laid in water-lime mortar, to be placed upon them.

The wall to be 2 feet thick, 28 feet long, and of such height as the levels of the engineer may indicate: said walls to be laid in a workmanlike manner, to the satisfaction of the engineer.

For the spans west of the 84 feet span, the foundations shall be prepared by driving, to a depth satisfactory to the engineer, seven good, sound, white oak, white or pitch pine, or cedar piles, 12 inches diameter.

Upon the foundation walls, and upon the foundation piles, timber bents are to be placed, constructed as follows: a sill and cap piece are to be framed upon seven vertical posts with the mortise and tenon joint. The posts to be of a sufficient length to make the top of the bent on a level with the bottom of the stringers. The posts are to be so placed that each shall be directly beneath its corresponding stringer.

The bents shall be stiffened by cross braces of plank pinned against the sides.

The bents placed upon the pile foundations shall have their sills fastened to the heads of the piles by the mortise and tenon joint, and by iron straps 3 inches by $\frac{1}{2}$ inch thick, passed around said sill and bolted to the piles. Four straps to each bent.

Superstructure for the 29 feet Spans.—See Plate 66.

The superstructure to be composed of seven stringers, laid at right angles upon the bents, and directly over the posts placed beneath.

Four stringers to be placed directly beneath the rails of the track.

With the exception of the middle stringer and post, braces are to be inserted, on a slope of 1 to 1, between the stringers and posts. The braces are to be notched in, and held by the mortise and tenon joint.

A railing, composed of cap piece and vertical posts, is to be fastened upon the exteriors of the outside stringers; and the stringers are to be floored over with 2½ inches thick pine plank. The quality of timber and style of workmanship to be precisely the same with the 84 feet span, before specified.

The railing posts, railing caps, and the exteriors of outside stringers, are to be painted with two coats of oil, of such colour as may be directed.

The whole of the above-named work, together with the delivery and acceptance of the materials, and all things connected therewith, to be under the direction and inspection of the said engineer.

Such parts of the above-named work as are not particularly described are to be in accordance with the directions of the said engineer.

Estimate of Timber for a double Track Bridge of 84 *feet Span, (3 trusses,)* 11 *feet in clear between trusses.—See Plate 64.*

No of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
6	Stringers or chords	White pine	61½	5 × 18	2767½
6	" "	" "	34½	"	1552½
6	Horizontal abutting pieces	" "	36½	5 × 15	1368¾
12	Sloping abutting braces	" "	30	"	2250
6	Minor horizontal abutting pieces	" "	18½	"	693¾
12	Minor sloping abutting braces	" "	21½	5 × 12	1290
12	Sloping braces at centre	" "	13	"	780
18	Arch pieces	" "	19	5 × 15	2137½
12	" "	" "	16½	"	1237½
9	Posts	" "	14½	5 × 12	652½
6	"	" "	11	"	330
6	"	" "	6¾	"	202½
6	Gallows frame posts	" "	19	"	570
2	Gallows frame caps	" "	26	7 × 12	364
8	Braces for ditto	" "	7½	6 × 12	360
12	Diagonal braces between posts	" "	15	5 × 12	900
6	" " "	" "	11½	"	345
6	" " "	" "	9½	"	285
14	Floor timbers	" "	26	7 × 12	2548
2	" "	" "	11⅛	"	156¼
2	" "	" "	40	"	560
4	" " braces	" "	12	7 × 9	252
6	Splicing pieces between stringers	" "	8	5 × 18	360
6	" " " "	" "	9	"	405
6	" " " sloping abutting braces	" "	7	5 × 15	262½
6	Splicing pieces between ends of arches	" "	4	"	150
	Flooring plank	" "	15	2½ × 12	4950
	Siding boards	" "	14½	1 × 4 to 10	6540
6	Railing pieces	" "	14	5 × 15	525
6	" posts	" "	6½	6 × 5	97½
6	Coping plank	" "	19	4 × 19	722
6	" "	" "	18	"	684
6	" "	" "	15	"	570
					36,868¾

Estimate of Iron for the above-named Bridge.

219 bolts of 1 inch round wrought iron, 15 inches in clear of head and nut.

heads and nuts, 2 inches square by ¾ inch thick.

washers for ditto, 3 inches diameter by ½ inch thick.

144 bolts for splicing bars, of 1 inch round wrought iron, 15¾ inches in clear of head and nut.

heads and nuts, 2 inches square by ¾ inch thick.

144 washers for ditto, 3 inches diameter by ¼ inch thick.

18 splicing bars of wrought iron, 11 feet long, 3 inches by ¾ inch thick.

6 wrought iron bands, 4 feet by 15 inches in clear, of 2½ by ⅝ inch thick iron.

6 wrought iron bands 2¾ feet by 15 inches in clear, of 2½ by ⅝ inch thick iron.

Total weight of iron, 3500 lbs.

54 white oak trenails, 8 inches square, 2 inches diameter, and 15 inches long.

30 white oak trenails, 8 inches square, 1 inch diameter, and 15 inches long.

Estimate of Timber for one Span of 29 feet in clear between Bents.—See Plate 66.

No. of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
	For one Bent.				
7	Posts	White pine	19	12 × 12	1596
1	Cap piece	" "	26	"	312
1	Sill	" "	"	"	312
2	Diagonal braces	" "	28	3 × 12	168
					2388
	Superstructure.				
7	Stringers	" "	30	7 × 14	1715
12	Braces for ditto	" "	15	7 × 10	1050
16	Railing posts	" "	4¾	5 × 5	158⅓
2	" pieces	" "	30	"	125
30	Flooring plank	" "	23$\frac{7}{12}$	2¼ × 12	1768¾
2	Coping "	" "	30	2½ × 8	100
					4917$\frac{1}{12}$
				Total . .	7305$\frac{1}{12}$
72	White oak trenails, 8 inches square, 1 diam., and 12 long				
24	White oak trenails, 8 inches square, 1 diam., and 7 long				

Note.—The subjoined estimate is founded on the prices contracted for, and shows the actual cost of one span.

Estimate of cost for one Span, exclusive of Foundations for Bent.

30 lineal feet of superstructure . . .	@ $ 5·00 . .	$150·00
2388 feet board measure timber in *one* bent	@ $15·00 ℔ M.	35·82
framing ditto	@ $30·00 „	71·64
		$257·46

Where foundation walls are used, add for		
6·22 cubic yards stone masonry @ $4·93 ℔ cub. yd.	$30·66	
435 feet B. M. plank in foundation @ $10·00 ℔ M.	4·35	
		$35·01

Where bearing piles are used, add for		
175 lineal feet of pile timber @ 3 cents ℔ foot .	$ 5·25	
Driving ditto @ 24 cents ℔ foot	$42·00	
Iron for ditto	$15·00	
		$62·25

Plates 67 and 67* represent a bridge of one span of 100 feet in the clear between abutments, constructed across the Mohawk River, near the city of Rome: it is the longest span used on this railroad.

The manner of trussing is precisely similar to the 84 feet span shown in Plate 64, making the obvious alterations required for an extended span: it rests on pile bent abutments.

The dimensions are figured on the plan, and the whole is so shown in detail as to preclude the necessity of further description.

The piles in the abutments are so arranged as to bring one pair of them directly beneath each truss.

Fig. 1 is a geometrical elevation of the side of the truss.

Fig. 2 is a horizontal section of one-half the bridge,

taken at the top of the stringers, showing the connexion of the floor beams and flooring.

Fig. 3 is a cross section taken at right angles through the centre of the bridge, and shows a side view of the abutment.

Fig. 4. is an isometrical projection of the truss.

Specification of a Bridge of 100 *feet Span, to be constructed for the Utica and Syracuse Railroad Company, over the Mohawk River, at Burrows, about* 5 *miles east of the city of Rome; reference being had to the plan of the same, in the office of the Engineer of the said Company.*

FOUNDATIONS.

The superstructure is to be supported by timber bents, or abutments formed by driving piles into the ground in the manner herein specified.

The bearing piles are to be of sound, square-edged white oak, free from sap or loose knots, and of a sufficient length to be driven into the ground to a depth satisfactory to said engineer, say, not to exceed 25 feet. After they are driven, the piles are to be sawed off at the proper elevation to receive the cap pieces. The cap pieces are to be fastened on by the mortise and tenon joint, and secured by white oak trenails. Each abutment is to consist of two rows of piles, separated by a distance of 3 feet, and driven in straight lines at right angles to the axis of the bridge. Each row is to consist of 7 piles. The cap pieces are to be of the same size and quality as the piles, and to be 27½ feet long. The sides

of the bents are to be planked up with sound 2½-inch pine planks, laid edge to edge, and pinned against the piles.

The bents to be filled with cobble stone, well packed.

The timber to be delivered upon the ground of the lengths and sizes specified in the annexed estimate of timber.

Estimate of Timber for one Abutment.

No. of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in lineal feet.	Amount in feet, board measure.
14	Piles	White oak	32	12 × 12	488	
2	Caps	.. ..	27½	..	55	
3	Cross caps	.. ..	5	..	15	
					558	
	Plank	White pine				750
46	White oak trenails, 8 in. square, 1 diam., and 12 long					

Estimate of Cost for one Abutment.

558 lineal feet of white oak timber @ 12 cents ꝑ lineal foot	$ 66·96
488 ,, ,, pile timber driven @ 75 ,, ,, ,,	366·00
70 ,, ,, cap ,, framed @ 25 ,, ,, ,,	17·50
750 feet, board measure, of pine plank @ $ 10·00 ꝑ M. B. M.	7·50
	$ 457·96

Note.—The above estimate is for the contracted prices, and shows the amount paid on the final estimate.

SUPERSTRUCTURE.

The superstructure is to consist of one stretch of 100 feet in clear of abutments, and to comprise three trusses, so placed as to leave a clear way of 11 feet between them. The trusses are to be framed and braced according to said

plan, and the timbers are to be of such dimensions as are marked thereon, and stated in the annexed estimate of timber.

The timbers are to be firmly bolted together with iron screw bolts. The whole is to be framed and braced in the most substantial and workmanlike manner: every shoulder and joint is to fit and bear with the utmost precision, and the timbers are to be placed in the positions shown on the plan, with the utmost accuracy.

The timber to be used must be sound, fair, and square-edged pine, free from sap, wane, and large or loose knots, and that has been sawed during at least six months. The floor is to be laid with sound $2\frac{1}{2}$ inches thick white pine planks, placed edge to edge, and spiked firmly to the floor beams. The floor beams to be laid at right angles to the direction of the trusses. The sides of the trusses are to be boarded up with sound, well-seasoned white pine tully boards, planed and matched together, placed edge to edge vertically, and firmly nailed to the trusses. Fuming pieces, 1 inch thick, to be interposed between the casing and trusses. A small space is to be left between the ends of the flooring plank and the sides of the casing. The trusses are to be coped with white pine plank that project 2 inches beyond the sides of the truss, and have their upper surfaces bevelled, to carry off the water; their under surfaces to be wrought with a groove. The timbers are to be notched over each other at the places shown on the plan; each notch is to be squarely cut of the exact size, and to fit and bear with the utmost precision.

Painting.—The sides and ends of the trusses, the gallows frames, and external braces, to be covered with two coats of oil and white lead.

Iron.—The screw bolts are to be round wrought iron, 1 inch in diameter, to be driven through $\frac{7}{8}$-inch auger holes. Their heads to be 2 inches square by $\frac{3}{4}$ inch thick, and to be firmly fastened on.

The nuts to be of the same dimensions with the heads, and each provided with a washer 3 inches in diameter by $\frac{1}{4}$ inch thick. Where directed by the engineer, white oak trenails, octagonal in section, and $1\frac{1}{2}$ inch in diameter, may be used.

The bands are to be made of the best wrought iron, $2\frac{1}{2}$ inches by $\frac{5}{8}$ inch thick, firmly welded or screwed together, of the size specified in the annexed estimate of iron.

One stringer in each truss shall be spliced once, and its opposite shall be spliced twice at such places as will equally break joints, as represented on said plan. There shall be 3 splicing bars to each joint. Each splicing bar shall be 11 feet long, of 3 inches by $\frac{3}{4}$ inch thick wrought iron, fitted upon the ends with spurs, and pierced by holes, as shown on said plan. The splicing bars to be firmly fastened to the stringers by 1-inch diameter screw bolts, of the quality specified above. The workmanship of the iron to be neatly and accurately executed, and the material to be of the first quality.

The whole of the above work, together with the delivery and inspection of the materials, to be under the direction of the engineer; and the said work is

to be constructed in every respect in accordance to the specifications and plans; subject to such alterations as may be directed from time to time by the engineer.

Estimate of Timber for a double Track Bridge, 100 *feet Span,* 11 *feet in clear between trusses.*

No. of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
3	Chords or stringers	White pine	50½	5 × 18	1136¼
3	.. ..	.. ..	59½	..	1338¾
3	.. ..	.. ..	63	..	1417½
6	.. ..	.. ..	23½	..	1057½
6	Horizontal abutting pieces	.. ..	54¾	5 × 15	2053⅛
12	Sloping .. braces	.. ..	30¾	..	2306¼
6	Minor horizontal abutting pieces	.. ..	18¾	..	703⅛
12	.. sloping .. braces	.. ..	21¼	5 × 12	1275
12	Sloping braces at centre	.. ..	16¼	..	975
36	Arch pieces	.. ..	18½	5 × 15	4162½
9	Posts for gallows frames	.. ..	19	..	1068¾
12	Posts	.. ..	14¾	..	1106¼
6	..	.. ..	11½	..	431¼
6	..	.. ..	7¾	..	290⅝
6	.. for railing	.. ..	5½	..	206¼
6	Caps ..	.. ..	10	..	375
18	Diagonal braces (between posts)	.. ..	15¾	5 × 12	1417½
6	.. .. ..	.. ..	12¾	..	382½
6	.. .. ..	.. ..	9¾	..	292½
3	Gallows frame caps	.. ..	26¼	7 × 12	551¼
12	.. .. braces	.. ..	6½	6 × 12	468
19	Floor beams	.. ..	26	7 × 12	3458
2	.. ..	.. ..	23½	..	339
3	.. ..	.. ..	42	..	882
6	Braces to ditto	.. ..	13½	7 × 9	425¼
9	Splicing blocks between stringers	.. ..	7¾	5 × 18	523⅛
6	.. .. ..	.. ..	6	..	270
6	.. .. sloping abutting braces	.. ..	5	5 × 15	187½
6	Splicing blocks between feet of arches	.. ..	4	..	150
6	Cap plank	.. ..	28	3 × 19	798
12	.. ..	.. ..	10	..	570
	Siding boards	.. ..	15	1 × 4 to 10	8000
	Flooring planks	.. ..	15 to 20	2½ × 12	6586
					45,203¾
120	White oak trenails, 8 inches square, 1½ diameter & 15 long				
33	White oak trenails, 8 inches square, 1 diameter, & 12 long				

Estimate of Iron.

294 bolts, of 1 inch diameter round wrought iron, 15 inches in clear of head and nut.

heads and nuts, 2 inches square by $\frac{3}{4}$ inch thick.

washers for ditto, 3 inches diameter by $\frac{1}{4}$ inch thick.

270 bolts for splicing bars, of 1 inch round wrought iron, $15\frac{3}{4}$ inches in clear of head and nut.

heads and nuts for ditto, 2 inches square by $\frac{3}{4}$ inch thick.

27 splicing bars of wrought iron, 11 feet long, 3 inches by $\frac{3}{4}$ inch thick.

6 wrought iron bands, 4 feet 2 inches by 15 inches in clear, of $2\frac{1}{2}$ inches by $\frac{5}{8}$ inch thick iron.

6 wrought iron bands, 3 feet 1 inch by 15 inches in clear, of $2\frac{1}{2}$ inches by $\frac{5}{8}$ inch thick iron.

Total weight of iron, 4857 lbs.

Plate 68 represents a trestle bridge constructed over the Onondaga Creek and Valley, for a length of 600 feet. It is at the western terminus of the Utica and Syracuse Railroad, and was constructed by that Railroad Company for the Syracuse and Auburn Railroad at their junction. The Onondaga Creek, at the place crossed by the line of railroad, was formed into a mill pond by an artificial dam below; so that the depth of water at the lowest was about 11 feet. The foundations were constructed during the winter season, while the pond was frozen over. The position of each pile having been accurately ascertained on the level surface of the ice, a square hole was cut just of the proper size to admit the pile; the piling machine was then brought on, and the piles were driven into the hard gravelly bottom below the bed of the pond.

The trestle-work was executed in spans of 30 feet

each, measuring from centre to centre of bents, and it very much resembles, in its general features, the trestle bridge over the Oneida Creek Valley, represented in Plate 66. It is, however, a much firmer structure. The piles and the sills of the bents are 14 inches square, and the braces that stiffen them are double on each side, and are held in position by screw bolts of iron in place of trenails. The heads of the piles were sawed off at the level of 1 foot above the surface of the ice, and the sills of the bents were fastened upon them by the mortise and tenon joint, and further secured by wrought iron straps; four straps to each bent. A general description of the superstructure would be but a repetition of that for the Oneida Creek Valley Bridge; it is therefore omitted. The subjoined specifications and estimates of material fully explain the kind and quality of workmanship and material used in its construction.

Specification for a Trestle Bridge to be constructed for the Utica and Syracuse Railroad Company, over the Onondaga Creek, near Syracuse, reference being had to the plans of the same, in the office of the Engineer of the said Company.

The bridge is to consist of 20 spans of 30 feet each, measuring from centre to centre of bents, and to be of a sufficient width to accommodate two tracks. To be constructed on pile foundations.

FOUNDATIONS.

The foundations to be prepared by driving bearing piles of square-edged, sound white oak timber, to the

depth of the hard bottom beneath the creek bed, or to the satisfaction of the engineer. The piles to be sawed off at a level of 1 foot above low-water mark, and to have their heads joined by the sill of the bent. The piles to be of straight and sound white oak timber, hewn square, and totally free from wane and large or loose knots, and to be of the dimensions specified in the annexed estimate of timber. The piles are to be seven in number for each bent, to be driven in a straight line, at right angles to the line of railroad, and at such distances apart as are represented on the said plan.

BENTS.

Upon the heads of foundation piles, a sill, 14 inches square and 27 feet long, is to be fastened by the mortise and tenon joint, and further secured by wrought iron straps, passed over them and bolted to the piles. Immediately over the piles, seven vertical posts, 12 inches square, are to be erected, of the length specified in the annexed estimate of timber. Upon them a cap, 12 inches square, is to be mortised and firmly held by white oak trenails.

Upon each side of the posts, two parallel braces, of the dimensions specified in the annexed estimate of timber, are to be firmly fastened by wrought iron screw bolts. All the workmanship of the foundations is to be of the first-rate style of joinery, and executed as per plans and specifications. The timber for the bents to be of sound, square-edged white pine timber, free from black or loose knots, windshakes, wormholes, and sap.

Iron.—There are to be four straps for each bent, and two screw bolts to each strap. They are to be of the dimensions stated in the annexed estimate of iron, and are to be manufactured from the best quality of American wrought iron.

A screw bolt is to be passed through each brace and post at their intersection. The screw bolt is to have a clear and strong thread cut at its end, 2 inches long, and to be manufactured from the best quality of American wrought iron.

SUPERSTRUCTURE.

The superstructure is to be composed of 7 stringers, laid across the bents, at right angles to the line of their direction, and directly over the posts placed beneath. The ends of the stringers to abut squarely, and to be notched one inch over the cap pieces of the bents.

A sloping brace is to be inserted between each stringer and post, and to be firmly held in position by the mortise and tenon joint, well trenailed.

The tops of the stringers are to be planked over with 2½ inches thick white pine plank, firmly spiked down. Upon the outside stringer a railing is to be erected, composed of railing posts and cap pieces. The cap pieces to be mortised upon the posts. Each joint of the cap piece is to be protected from water by a piece of canvass, saturated with white lead and oil, tightly bound around it. A projecting coping plank, placed upon the level of the flooring planks, is to be notched around the posts, and firmly spiked upon the outside stringer.

All the timber used is to be of the first quality white pine, free from sap, wane, windshakes, wormholes, and all loose or large knots; to be sawed perfectly square-edged, and true to the sizes stated in the annexed estimate of timber; and to be framed and braced in the most substantial and workmanlike manner. Every shoulder and joint is to fit and bear with the utmost precision, and to be united by white oak trenails of the specified size and section. The timbers are to be placed in the positions represented on the plans with the utmost accuracy, to the satisfaction of the engineer.

Painting.—The railing posts, cap pieces, coping planks, and the exteriors of outside stringers, to be covered with two coats of oil and white lead.

The delivery and acceptance of all the materials used for the said bridge to be under the inspection of the said engineer, and to be executed to his satisfaction. The plans to be subject to such slight alterations as he may from time to time direct.

Estimate of Timber for one Span of 29 feet in clear between Bents.—See Plate 68.

No. of pieces.	Names.	Kind of timber.	Length in feet.	Size in inches.	Amount in feet, board measure.
	For one Bent.				
1	Sill	White pine	27	14 × 14	441
1	Cap piece	.. ..	..	12 × 12	324
7	Posts.	.. ..	13	..	1092
4	Braces for ditto	.. ..	25	3 × 8	200
					2057
	Superstructure.				
7	Stringers	.. ..	30	7 × 14	1715
14	Braces for ditto	.. ..	16½	7 × 10	1347½
15	Railing posts	.. ..	4¾	6 × 6	213¾
2	.. caps	.. ..	30	..	180
60	Flooring plank.	.. ..	12	2½ × 12	1800
2	Coping	.. ..	30	2¼ × 9	112½
					5368¾
28	White oak trenails, 8 inches square, 1 diam., and 14 long				
42	White oak trenails, 8 inches square, 1 diam., and 12 long				
28	White oak trenails, 8 inches square, 1 diam., and 7 long				
30	White oak trenails, 8 inches square, 1 diam., and 6 long				

Estimate of Iron.

4 straps of 2⅝ × ⅝ inch thick wrought iron, 7 feet long, pierced with 4 holes, opposite; for 2 bolts.

8 screw bolts of 1 inch round wrought iron, 15½ inches long in clear between head and nut.

heads and nuts, 2 inches square by ¾ inch thick.

20 screw bolts of 1 inch round wrought iron, 15 inches in clear between head and nut.

heads and nuts, 2 inches square by ¾ inch thick.

washers, 3 inches diameter by ¼ inch thick.

2 screw bolts of 1 inch round wrought iron, 18 inches in clear between head and nut.

heads and nuts, 2 inches square by ¾ inch thick.

washers, 3 inches diameter by ¼ inch thick.

Total weight of iron, 298 lbs.

Estimate of Cost for one Span, inclusive of Foundations for Bent.

224 lineal feet of white oak pile timber, 14 inches square, @ 3 cents ℘ foot	$	6·72
Driving ditto, @ 35 cents ℘ lineal foot		78·40
298 lbs. of wrought iron @ 12 cents ℘ lb.		35·76
30 lineal feet of superstructure, @ $ 5·08 ℘ foot		152·40
2057 feet, board measure, white pine timber in one bent, at $ 15·00 ℘ M. B. M.		30·86
Workmanship for ditto, at $ 30·00 ℘ M. B. M.		61·71
	$	365·85

Note.—The above estimate is founded on the prices contracted for, and shows the actual cost for 1 span.

JOINERY.

Plate 69 represents the various joinery used in framing the bridges. The timbers and proportions given in this Plate for the joinery of the truss bridges are made out for the 84 feet span; but they are of the same character in all the trusses, the proportions being slightly altered to suit the increased or diminished section of timbers. The joinery for the trestle bridges is for both valley crossings.

Fig. 1 represents the extremity of a stringer, notched to receive the main abutting braces and the arch pieces. The notches are cut to a depth of 3 inches, at right angles to the inclination of the braces. The ends of the braces and arch pieces are then cut so as accurately to fill the notches; and being placed in position, the iron bands of $2\frac{5}{8}$ by $\frac{5}{8}$ inch wrought iron (represented in fig. 2) are driven over them, and they are further secured

in position by inserting bolting blocks between them and their opposites, and firmly bolting through.

Fig. 3 represents the vertical posts of the bridge truss, with a diagonal brace inserted between them. The posts have a double notch cut at either extremity upon opposite sides, to a depth of 3 inches at right angles to their longitudinal axis. The notches are cut at such distances from the ends of the posts that they shall be covered by the main abutting pieces and stringers; thus preventing them from falling out sideways, as they retain their places by virtue of their position alone.

Fig. 4 represents the coping plank for the bridge trusses. It is bevelled as shown in fig. 4, and is grooved upon its under side. The siding boards that case the truss are nailed on, flush with the outside of coping plank; between them and the truss frames fuming pieces 1 inch thick are interposed, over which space the groove is made, to further insure the exclusion of water from the truss frame.

Fig. 5 represents the joinery of the gallows frames. A pair of opposite posts (one in each truss frame) are run up to a sufficient height, and a tenon 4 by 5 inches is framed upon their tops. A gallows frame cap, with its extremities mortised and notched 2 inches over the posts, is then placed upon them and fastened down by trenails. At a distance of 5 feet from the post a mortise and notch are made in the under side of the gallows frame cap, and a brace tenoned to match it is inserted between the cap and post: the lower extremity of the brace rests upon the main abutting piece, and against the side of the post,

to which it is fastened by a screw bolt. The gallows frame brace is notched 2 inches on one extremity at right angles to its inclination, and is fitted accurately to the step cut in the cap to receive it. The tenon of the brace is secured by a trenail.

Fig. 6 represents the framing of the floor timbers: a notch 1 inch deep and 15 inches long, or of equal length with the thickness of the truss frame, is made upon its under side, and the floor timber is simply laid over the stringers without any fastening.

Fig. 7 represents the joinery of the long floor timbers and floor timber braces. The floor timber is notched 1 inch over the stringer, and being produced to a sufficient length, a step and mortise, as shown in fig. 7, are made at a distance of 6 inches from the end. From the extremity of the floor timber, a brace is inclined against the truss, to assist in preserving it in a vertical position. The brace, at one extremity, is tenoned, and fitted with a notch 2 inches deep at right angles to its longitudinal axis. The notch and tenon fit accurately into the step and mortise of the floor timber, where they are secured by trenails. The other end of the floor timber brace is notched so as to embrace the main abutting piece of the truss frame, to which it is fastened by screw bolts or spikes.

Fig. 8 represents in detail the railing arrangement for a trestle bridge. The railing post, at its lower end, is notched out half its width for a length of 9 inches; it is then placed upon the stringer, and its descending part is firmly spiked against the side. A coping plank, with notches 6 inches square, to embrace the posts, is then

spiked upon the top of the stringer. Its inner edge is placed flush with the inner face of the posts, and its outside edge projects 2 inches beyond their outside faces. The tops of the posts are formed into a tenon which fits into a corresponding mortise in the cap, and is firmly held there by trenails.

Fig. 9 represents the mortise and tenon joint that unites the vertical posts, caps, and sill pieces, in the trestle portion of the abutment shown in Plate 65. The mortise and tenon joint, here represented, is always the same in character; the size of the tenon being only varied to suit the section of timber that contains it; it is $4 \times 5 \times 12$ inches. The cap is notched down upon the post 1 inch, and is mortised of the exact size of the tenon, which is then fastened with 2 trenails.

Fig. 10 represents the joinery of the cross ties used in the same trestle (Plate 65). They are dovetailed 4 inches of their depth into the caps and sill pieces of the vertical frames; the dovetails are 8 inches broad at their narrowest end: they are tightly fitted into hollows cut into the caps and sill pieces, of their exact size and shape, and are firmly held by 3 trenails to each joint.

RAILWAY SUPERSTRUCTURE.*

The plan of superstructure adopted on the Utica and Syracuse Railroad differs widely in its detail from any

* This portion of Mr. Isherwood's description of the mechanical works on the Utica and Syracuse Railroad is so closely connected with the subjects detailed in the preceding pages, that it has been deemed expedient to insert it in this Supplement.—J. W.

before contrived in America. The timbers are of greater section, a larger amount of iron is used, and greater care was taken in its adjustment than has been generally practised on roads constructed with the plate rail.

PILE ROAD.

As a considerable length of this road passes through a deep swamp, a foundation of great permanency was required: this gave rise to a modification of the superstructure, and formed that which is known as pile road. The swamp varied in depth from 10 feet to 60 feet, and was nearly on a dead level throughout; the grade line closely corresponded with its surface, so that it was necessary to reach the hard bottom before *any* foundation could be effected. Piles were adopted as the cheapest and most efficacious means to secure a durable and substantial basis; they were driven to their places by a steam pile driver (Cram's Patent).* This was a machine formed of a platform about 25 feet long and 8 feet broad; at one end was erected two pair of leaders or guides, in which the hammers moved. Immediately behind the leaders were fixed the rollers, with the necessary brakes and gearing for working the hammers, raising the piles, &c. The rollers were revolved by a small high-pressure steam engine, occupying the rear of the machine. The arrangement of the leaders was the same as in ordinary piling machines; a curved piece of wood forced open the shears when the hammers reached

* I have added to the work the engraved Plate of a pile-driving steam engine, which, although it shows its operation for driving piles for a railway, is equally applicable to pile-driving in bridge constructions.—J. W.

their elevation. The hammers were confined to the leaders by a groove; they weighed about 1000 lbs. each, were made of cast iron, and at their last blow fell through a space of 27 feet. A pair of piles were driven at one operation by this machine; when driven, cast iron rollers were placed upon their heads, and the machine, by means of an inverted rail, moved on to the next place. The heads of the piles, sawed off to reduce them to the proper level, were found sufficient to supply the furnace with fuel.

The men employed in operating the machine were a foreman, a steam engineer, two brakemen, and two men in front at the saws; also a horse and cart to furnish water for the boiler. Properly geared in front of the machine, and between the leaders, was a saw that played on a sway bar and could be pressed against either pile as it was driven home. The machine was manufactured complete for the cost of $2000. The pile was prepared for being driven by simply sharpening one end to a point, and squarely butting the other; it was drawn up by ropes worked by the engine, secured in position between the leaders, and driven to the hard bottom. Generally the piles manifested no disposition to split; where they did, their heads were encompassed with an iron hoop. When the pile was not of sufficient length to reach the hard bottom, another was dowelled upon its head, and this was repeated as often as necessary. The piles were charred to increase their durability, and an auger hole, bored in their heads for the purpose, was filled with salt, and securely plugged up. The piles

were driven 4·98 feet from centre to centre transversely, and 5 feet from centre to centre longitudinally. When the piles were driven to the satisfaction of the engineer, the superstructure was framed upon them as follows: (see Plates 70 and 70^b). A cedar cross tie, 4 by 12 inches in section and 8 feet in length, was securely fastened upon the heads of each pair of piles by a white oak trenail, 2 inches in diameter, 16 inches in length, and octagonal in section. At right angles to, and upon the cross ties, were laid the white pine longitudinal rails: they were fastened to the cross ties by cast iron knees, spiked against both sides of the rail, and down upon the cross ties; two knees at each intersection of tie and rail. When the rails joined a knee of larger size was used. The joining of the rails was effected by simply bringing their square ends into close contact. The single knee is shown in Plate 70^c, fig. 3. The double knee is shown in fig. 4. The rails were 15, 20, 25, and 30 feet in length, 8 by 8 inches in section: at each end of the rail an auger hole was bored, filled with salt, and plugged up, to prevent the premature decay that naturally takes place at the joints. Upon the tops of the rails, and precisely over their centres, were placed white oak ribbons, 3 by 1¼ inch in section; upon them, with their inner edges lying in the same vertical plane, was placed the iron plates, 2½ by ⅝ inch in section: both ribbon and iron plate were then firmly spiked to the rail. The iron plates were separated by a distance of 4·73 feet between them, being the width of the track in the clear. The iron plate was properly laid with end plates, and all the details and fixtures were as shown in Plate 70^a.

Specifications of the Materials and of the manner of constructing the Pile Road for the Utica and Syracuse Railroad.

PILES.

The piles are to be of perfectly sound white, yellow, or pitch pine, white elm, cedar, black ash, tamarack, or hemlock timber.

They are to be straight, and no piles having marks of decay will be accepted.

The piles are to measure at the butt, or larger end, not less than 12 nor more then 17 inches in diameter, exclusive of bark, and to be of the length specified in the requisitions for the same.

The piles are to be delivered along the line of the road at such stations as the requisitions shall designate, so as to be conveniently used by the piling machine: they are to be driven into the hard bottom, to the satisfaction of the engineer, by a hammer weighing 1000 lbs., and falling through 27 feet at the last blow. The piles are to be driven perpendicularly, 4·98 feet from centre to centre transversely, and 5 feet from centre to centre longitudinally: they must be sawed or chopped off squarely at the butt, well sharpened to a point at the smaller end, and if there should appear to be any danger of splitting, the heads are to be bound with iron hoops.

In such cases where the pile shall not be of sufficient length to reach the hard bottom, it shall be sawed off square, and a hole 2 inches in diameter and 12 inches deep shall be bored into its head: into this hole a white

oak trenail, 2 inches in diameter and 23 inches in length, shall be driven; and another pile, similarly bored, shall be placed upon it, brought to its proper position, and driven home to the satisfaction of the engineer.

The piles must be accurately driven, with regard to position and vertical direction, and if any pile shall be found so much out of place that the longitudinal rail would not cross it, it must be sawed off, and another driven by its side. Smaller inaccuracies may be obviated, if so directed by the engineer, by inserting a brace between the pile and cross tie; and all loss or waste of material, so caused by the carelessness of the contractor or his agents, shall be at the expense of the contractor. When driven to the required depth, the piles are to be cut off on the grade line, by circular saws, carefully adjusted to it; and if any pile shall be sawed off so unevenly as to require extra labour to fit the superstructure upon it, the engineer shall estimate the expense of such labour, and deduct it from the prices of the pile contractor, unless said contractor also lays the superstructure.

CROSS TIES.—SEE PLATE 70.

The cross ties are to be square-edged sawed planks, 12 inches by 4 inches in section, and 8 feet long, sawed off square at the ends, and clear of stub-shot. They are to be of white cedar timber, perfectly free from wane, sap, large or loose knots, and other imperfections. There will be one over each pair of piles. The centre of the tie is to be on a line with the centre of the track. The

ties are to be neatly fitted upon the piles, and an auger hole, 1⅞ inch in diameter, is to be bored through the tie, and 12 inches deep into the head of the pile: into this hole a white oak pin, 16 inches long, 2 inches in diameter, and octagonal in section, is to be driven. When it is driven within 3 inches of the hole, its head must be split, and a wedge inserted, when it shall be driven home.

The pins must be split from straight-grained, well-seasoned white oak timber, and wrought with a plane to the specified size and shape.

RAILS.—SEE PLATE 70[a].

The rails are to be sawed on four sides, true and even, so as to be in section precisely 8 inches square, and in lengths of 15, 20, 25, and 30 feet, exclusive of stub-shot: one-half of the rails is to be 20 feet long or over.

The rail timber is to consist of perfectly sound, square-edged white and yellow pine, free from wane, shakes, sap, and black or loose knots. The timber is to be inspected, on or after delivery, by the engineer, who shall have full power to reject every stick that is in his opinion unfit for use, or contrary to this specification. The rails are to be laid longitudinally on the cross ties, and directly over the centre of the piles beneath: they shall be separated by a distance of 4·65 feet between them. At each intersection of the rail and cross tie two cast iron knees shall be spiked in the angles, so as firmly to secure the rail upon the cross tie. The rails shall be laid so as to break joints along the length of the road.

The joining of the rails shall be effected by simply bringing their square ends in contact, and holding them in position by a double knee, of larger dimensions than the intermediate ones.

Near the ends of each rail auger holes shall be bored, filled with salt, and securely plugged up.

OAK RIBBON AND IRON PLATE.

Directly over the centre of the rails shall be laid white oak ribbons, 3 inches by $1\frac{1}{4}$ inch in section, and of lengths varying from 15 to 30 feet: at least one-half is to be 20 feet or over in length. They shall be so laid as to break joints with the rails beneath them. They shall be sawed on four sides, so as to be precisely of the above-mentioned section. They must be of the first quality white oak timber, free from sap, shakes, and loose or black knots. They shall be laid by merely bringing their square ends in close contact.

The iron plate shall be $2\frac{1}{2}$ inches by $\frac{3}{4}$ inch in section, and in lengths of 15 feet, weighing 30 tons to the mile. They shall be laid with their inner edges corresponding with the inner edges of the ribbons, so as to make a distance of 4·73 feet (the width of the track) between them. In laying them care shall be taken that no joint of the iron plate shall be within 5 feet of the joints of the rails or oak ribbons beneath.

The iron plates intended for the curved portions of the road must be bent by some effectual process, to suit the curves, before being laid. At each joint of the iron plate, end plates shall be neatly fitted into the oak

ribbons, so as to bring their upper surfaces in the same horizontal plane.

The end plates shall be of wrought iron, 6 inches long, $2\frac{1}{2}$ inches broad, and $\frac{1}{4}$ inch thick, fitted with a hole in each end, corresponding with holes in the ends of the iron plates. A space of $\frac{1}{4}$ inch shall be allowed for the variation of metal, in laying the iron plates.

When the iron plates, end plates, and oak ribbons are carefully adjusted, they shall be firmly fastened down together upon the rails, with pressed spikes, 6 inches long and $\frac{3}{8}$ inch square, one in every 18 inches.

KNEES.—SEE PLATE 70ᵃ.

The knees are to be single and double, made of cast iron. A double knee is to be placed at each joint of the rails. The single knee shall weigh $1\frac{1}{2}$ lb., and be fastened to the rail and cross tie by one spike driven into each. The double knee shall weigh $1\frac{9}{10}$ lb., and be fastened to the cross ties, and to the rails at their joints, by one spike in each rail, and by two spikes in the cross tie. The knee is to be placed upon both sides of the rail, two for each intersection of rail and cross tie. The spikes used in fastening to be pressed, 5 inches long and $\frac{5}{16}$ inch square.

Estimate of Cost for one mile of Superstructure for Pile Road, founded on the average of prices for which it was executed.

Amount.		Price.	$	cts.
56320	Feet, board measure, white and yellow pine rails, delivered at	$ 14·12 ℔ M. B. M.	795	24
1056	White cedar cross ties, delivered at . .	41 cents each	432	96
3300	Feet, board measure, white oak ribbons, delivered at	$ 25 ℔ M. B. M.	82	50
30	Tons of iron plates, delivered at . . .	$ 75 ℔ ton	2250	
720	lbs. of end plates, delivered at	9 cents each	64	80
6758	lbs. of cast iron knees, delivered at . .	5½ cents ℔ lb.	371	69
1500	lbs. of pressed spikes, to fasten on rail plates, delivered at	9 ..	135	
1089	lbs. of pressed spikes, to fasten knees to rails and cross ties, delivered at . .	9 ..	98	01
2112	White oak trenails, to fasten cross ties to piles, delivered at	1 cent each	21	12
	Salting and charring piles		50	
	Workmanship, putting timber together, and spiking on iron plate		350	
	Add for contingencies		50	
	Total cost for one mile of superstructure		4701	32
	Average cost of piling timber, and for driving the same per mile		1864	48
	Grand total per mile		6565	80

The total distance piled was $19\frac{26}{100}$ miles, and consumed an aggregate amount of 800,000 lineal feet of pile timber, at an average cost of $2\frac{1}{2}$ cents per foot.

GRADED ROAD.

The Utica and Syracuse Railroad, for the remaining distance of $33\frac{45}{100}$ miles, was graded in the usual manner by excavations and embankments; and being brought to the grade line throughout, a superstructure of the following description was laid upon it. A trench was excavated of the proper size, and a sill was firmly bedded in it. Where the sills abutted end to end, they were supported by a piece of wood of the same section laid beneath them. At right angles to, and upon the upper surfaces of the sills, were spiked the cross ties;

and again, at right angles to the cross ties, and immediately over the sills, were laid the longitudinal rails. The centre of the rail and sill are in the same vertical plane. Upon the longitudinal rails oak ribbons and the iron plates are firmly spiked. The rails are fastened to the cross ties by cast iron knees, and the detail in all respects is the same as described for the superstructure of the pile road.

This plan of superstructure is represented isometrically in Plate 70^b, and by a geometrical cross section in Plate 70^a.

The average price per cubic yard for earth excavation was $11\frac{75}{100}$ cents.

The average price per cubic yard for embankment was $11\frac{55}{100}$ cents.

Specifications of the Materials and manner of constructing the Superstructure for the Graded portion of the Utica and Syracuse Railroad.

SILLS.

The timber for the sills is to consist of sound white pine or hemlock timber, free from wane, shakes, sap, and large or loose knots, sawed on four or two sides. If sawed on two sides, it must make a stick 4 inches thick, and the sawed surfaces must be wide enough to measure 6 inches on each side of a line drawn over their centres. If sawed on four sides, it must make a stick precisely 4 inches thick and 12 inches broad. Said sill timber is to be delivered in lengths of 15, 20, 25, and 30 feet, and at least one-half is to be 20 feet long or over. A trench is to be excavated of the proper size, into

which the sills are to be firmly bedded: their upper surfaces must correspond exactly with the grade line. The sills shall be supported at their joinings by a block of pine wood, 4 inches thick, 12 inches broad, and 2 feet in length, placed under their ends.

The sills are to be placed at a distance of 4·98 feet from centre to centre.

CROSS TIES.

The cross ties are to be square-edged sawed planks, 8 by 4 inches in section, and 8 feet in length. They are to be of white cedar timber, perfectly free from wane, sap, large or loose knots, and all other imperfections. They are to be placed 5 feet from centre to centre on the centre line of the track. The cross ties shall be truly at right angles to the direction of the track, and shall be firmly spiked upon the sills by pressed spikes, 6 inches long and $\frac{3}{4}$ inch square. The centre of the cross ties shall correspond with the centre of the track.

RAILS, OAK RIBBON, AND IRON PLATE.

The rails, oak ribbons, and iron plate shall be precisely the same in kind and quality of materials, and the workmanship shall be of the same degree of excellence as specified for the superstructure of the pile road. All the details and finishing shall be precisely as stated in the specifications for the pile road superstructure.

Estimate of Cost for one mile of Superstructure for the Graded Road, founded on the average of prices for which it was executed.

Amount.		Price.	$	cts.
56320	Feet, board measure, white and yellow pine rails, delivered at	$ 14·12 ℔ M. B. M.	795	24
42240	Feet, board measure, white pine or hemlock sills, delivered at	$ 7·92 ..	334	54
1056	White cedar cross ties, delivered at . .	31 cents each	327	36
3300	Feet, board measure, white oak ribbons, delivered at	$ 25 ℔ M. B. M.	82	50
30	Tons of iron plates, delivered at . . .	$ 75 ℔ ton	2250	
720	lbs. of end plates, delivered at	9 cents each	64	80
6758	lbs. of cast iron knees, delivered at . .	5½ cents ℔ lb.	371	69
1500	lbs. of pressed spikes, to fasten on rail plates, delivered at.	9 ..	135	
421	lbs. of pressed spikes, to fasten cross ties to sills, delivered at	9 ..	37	89
1089	lbs. of pressed spikes, to fasten knees to rails and cross ties, delivered at . .	9 ..	98	01
	Workmanship, laying timber, and spiking on iron		400	
	Add for contingencies		100	
	Total cost for one mile		4997	03

CROSSING PLATES.

Plates 70^c and 70^d represent two modifications of crossing plates.

Fig. 1 shows one that diverges in the ratio of 1 in 7.

Fig. 4 shows one that diverges in the ratio of 1 in 4.

They diverge in various ratios, from 1 in 4 to 1 in 20, according as the rails cross each other at a greater or less angle: the general arrangement is the same in all: of course their lengths increase or decrease in an inverse ratio to the rate at which they diverge. The condition that governs their length is this, that they shall measure 3 inches across the tongue at one end, and 3 inches across the opening at the other.

These crossing plates are placed at every intersection of the rails, caused by one track crossing another ob-

liquely, or by one track, as in a branch, diverging from another.

They are made of cast iron, and are firmly spiked to a heavy sill of wood.

At the place where the greatest wear occurs, from the friction of wheels passing over it, steel plates are let into the iron, and, by their greater hardness, effectually resist the increased strain.

Fig. 1 shows a horizontal projection of a crossing plate that diverges 1 in 7.

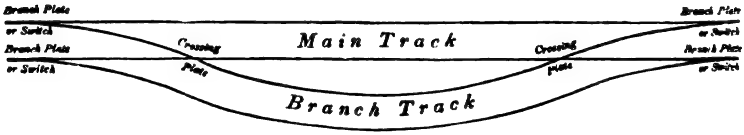
Fig. 2 shows a side view of the same.

Fig. 3 shows an isometrical projection of the same.

Fig. 4 shows a horizontal projection of a crossing plate that diverges 1 in 4.

Fig. 5 shows a side view of the same.

Fig. 6 shows a horizontal projection of the same.



BRANCH PLATES.—SEE PLATE 70^e.

The first four figures of Plate 70^e represent a branch plate or switch. A pair of these branch plates are placed wherever one track diverges from another, so that by simply shifting the slide, a train may be passed upon either track.

Their length is of course regulated by the degree of curvature with which one track sweeps from another: the condition that governs this is, that the space between

the iron plates at their termination shall measure $2\frac{1}{2}$ inches.

One end of the branch plate revolves on a pivot, and the other is moved by a wrought iron bar, worked by a lever, placed at a convenient distance: this iron bar is notched to admit the branch plates (the pair being moved together). The plates at each end of the slide make a proper finish, and secure its more perfect action: they have a jog at one end to receive the iron plate of the superstructure which is spiked firmly upon them.

The end plates are well spiked to a heavy wooden sill.

Sliding pieces are neatly let into this sill, and are kept well oiled, to facilitate the action of the slide.

The slide, end plates, and sliding pieces are of cast iron.

CULVERTS.

The last four figures of Plate 70^f represent a culvert of 4 feet chord. The subjoined specification sufficiently explains the manner of construction and the quality of the materials. Fig. 5 is an end view. Fig. 6 is a longitudinal section through the centre of the culvert. Fig. 7 is a ground plan of the wings and part of the trunk. Fig. 8 is a cross section of the trunk.

The culverts and viaducts of stone resemble each other very closely in the arrangement of the wings and in the general design: they were all constructed on timber foundations and arched, except the small square drains. It was therefore deemed unnecessary to present more than two; one of the larger and one of the smaller class.

The specifications of materials and construction were the same for all.

SPECIFICATIONS.

Foundations.—A pit shall be excavated, of suitable width and length, to the firm gravel. If this should occur at too great a depth below the bed of the creek, the pit shall be refilled with gravel, well puddled to a proper level. Upon this shall be laid a double course of timber, disposed at right angles to each other: the courses shall be of the thickness directed by the engineer, not to be less than 4 inches, nor to exceed 8 inches. The timber to be of first quality pine, or hemlock, free from any symptoms of decay.

In culverts over 4 feet chord the courses that compose the foundations for the wings shall be pinned or spiked together. The top of the planking shall be placed one foot beneath the bed of the stream, and shall be carefully puddled over between the abutments with 1 foot thick of clean gravel, to preserve the timber from rotting, when occasionally exposed to the action of the air.

Abutments.—The abutments shall be carried up plumb, on front and rear from the foundations to the springing line of the roofing arch: they shall not be less than 2 feet thick, and 2 feet high between the foundations and the springing line of the roofing arch, for culverts of 4 feet chord and under. For culverts over 4 feet and not greater than 8 feet chord, the width of the abutments shall not be less than 3 feet, and their height between the foundation and springing line of the upper

arch shall not be less than 4 feet. For culverts over 8 feet chord the abutments shall have a suitable increase, as may be directed by the engineer.

The face work shall be laid up in regular courses of hammer-dressed stone, and the backing shall be composed of rough-hammered masonry, well bonded into the face work.

For all culverts of 4 feet chord and under the courses shall not be less than 6 inches. For all culverts over 4 feet chord the courses shall not be less than 8 inches.

The ends of the abutments to conform to the wings.

Arches.—The arches are to be a semicircle in form.

For all culverts of 4 feet chord and under the arches shall be 1 foot thick.

For all culverts over 4 feet and not greater than 8 feet chord, the arches shall be 1½ foot thick. For culverts over 8 feet chord, and for culverts of lesser chord, when placed beneath the pressure of heavy embankments, the arches shall be increased in thickness, as may be directed by the engineer.

The thickness of the courses shall not exceed their depth, and in no case shall they exceed 18 inches. For culverts of 4 feet chord and under no course shall be of a less thickness than 6 inches, measuring on its face. For culverts over 4 feet chord no course shall be of less thickness than 8 inches, measuring on its face. The intrados of the arch to be well hammered to the proper curve. The beds of the arch-stones to be well hammer-dressed in the line of radii to the curve. The extrados

of the arch to be spalled off tolerably true and even to the line of the curve.

The ends of the arches to conform to the wings.

The arch-stones to break joints along the length of the culvert.

The square drains are to be roofed with flagging, 9 inches thick.

Spandrel Backing.—The spandrel backing is to be carried up plumb, 1 foot high above the rear corner of the abutment, for culverts of 4 feet chord and under, and 2 feet high for culverts over 4 feet chord. From thence it shall be carried on a line tangent to the exterior curve of the arch. The backing to be composed of rough-hammered stone, well laid and bonded.

Wing and Parapet Walls.—The wing and parapet walls shall be constructed on a semicircle, to which the ends of the abutments and arches shall conform. The face shall be carried up plumb: the rear shall be carried up plumb to a certain point, where a bevel extending down from the coping shall intersect it. The parapet walls shall have whatever elevation above the arch the engineer may direct.

The wing walls shall terminate in a rectangular buttress with a flat top, of such dimensions as the engineer may direct. From the inner side of the buttress the wing and parapet walls shall slope upward on whatever bevel the engineer may direct. The buttress, wing, and parapet walls shall be coped with coping stone, neatly jointed and dressed: for all culverts of 4 feet chord and

under the coping shall be 6 inches thick: for all culverts over 4 feet chord the coping shall be 9 inches thick.

The coping shall have a projection of 2 inches over the face of the walls.

The wing walls for culverts of 4 feet chord and under shall not be less than 2 feet thick at their foundation; and for culverts over 4 feet and not greater than 8 feet chord the wing walls at their foundation shall not be less than 3 feet thick; and for culverts of greater chord they shall be suitably increased, as directed by the engineer.

Whenever it shall be necessary to construct the culvert askew, the curve of the wings shall be modified accordingly, in a manner suited to the directions of the engineer.

The masonry shall be of the same character as specified for the abutments.

The stone for the masonry of culverts shall be of a sound and durable quality: the stretchers shall be in breadth at least 1½ their depth, and in no case less than 1 foot. The headers for the culverts of 4 feet chord and under shall extend through the abutments and wing walls; and for culverts of greater chord they shall be at least 2½ the depth of the course in length. The beds shall be dressed back 6 inches from the face, and laid horizontally with a full bearing of water-lime mortar. The headers shall be properly interspersed to make a firm and substantial bond.

Plate 70[e] represents a viaduct of 14 feet chord,

constructed at Lodi, beneath the Erie Canal. The wings are arranged askew 30°. It furnishes the best example of the larger class of culverts and viaducts, the others differing from it chiefly in the local variations. The work was executed with great neatness and precision. The courses were composed of large stone, neatly dressed, laid, and pointed. The quality of materials and the style of workmanship in all respects accord with the foregoing specifications.

Fig. 1 represents the ground plan of the wings and part of the trunk, showing the arrangement of the timber foundation.

Fig. 2 is a longitudinal section through the centre of the viaduct, and is supposed to be viewed in right-angled directions to the centre.

Fig. 3 is a cross section, taken at right angles through the trunk.

Fig. 4 is an end view, seen at right angles to the front line of the wings.

DESCRIPTIONS OF THE PLATES.

PREFACE

TO

DESCRIPTIONS OF THE PLATES.

THE irregular character of these descriptions appears to require some explanation. It will be found that in certain cases sufficient details and particulars of construction have been given to comprise rather a history of the works themselves, and the mode of their execution, than a mere description of the Plates. In other cases, again, all that has been presented is a description derived from the Plate. With respect to the former class of descriptions, we have great pleasure in acknowledging the valuable information furnished by the engineers of the works—information which has enabled us to execute with satisfaction the more perfect of the descriptions. At the same time it would be unjust to omit all expression of thanks to the engineers of those works which we have not had it in our power to describe in detail. With almost a single exception, we have received repeated assurances of their readiness to afford every possible information. The only discouraging circumstance connected with this general good feeling has been the delay which has attended the performance of the numerous promises it gave rise to. At length, so formidable had this delay become, and withal so unjust to the whole body of subscribers, that, in preference to any further procrastination, it was thought expedient to compile into one mass all the scattered information which had been received, and to reserve for a future edition such particulars as are necessary in certain cases to render the descriptions complete.

Whilst the full and detailed descriptions are comparatively the only ones which will possess much interest for the actually practising engineer, and for the more advanced student, we are yet anxious to urge upon the more youthful members of the profession the importance even of the dry descriptions of the Plates. It may be that these descriptions often contain nothing more than what an attentive study and examination of the Plate

would have suggested to the student himself. At the same time, however, they will have the effect of primarily drawing his attention to the study of the Plates; and we fear not to assure him that, without exception, every example in this work is well worthy an attentive and systematic examination. By merely turning over the Plates of a work like this, comparatively nothing more can be learnt than the general features of the design. The true study for the engineer lies in the contemplation of these fine Plates as a kind of text, affording at all times that confidence in future operations which arises from the fact that all they contain has been actually executed, and, for the most part, with perfect success.

The great variety of ingenious expedients and combinations which they exhibit in stone, wood, iron, and wire, will lose half their value, and convey scarcely any instruction, to him who hastily and carelessly hurries through them. On the other hand, where a diligent and careful study is bestowed, they will be found second only in value and importance to that practical information which is afforded by actual employment on engineering works.

If we might venture to give expression to a single sentiment which implies a somewhat sweeping reproach, we should say that, amongst the profession generally, the valuable additions made to the engineer's library by the publications of the last few years appear to have been not sufficiently appreciated. Every profession—law—divinity—medicine—each has its standard works, and amongst them all the most valuable are those which record facts and register examples. So for the engineer, the most useful study must ever be the practical record of existing combinations. In a physical profession like his, in which things and not words are to be dealt with—material elements, and not aërial fancies or ideas—it is chiefly by a knowledge of these combinations that he is able to go on from age to age increasing and improving his resources, and extending the sphere of his operations over every part of the earth.

DESCRIPTION OF THE PLATES.

CENTERING FOR BALLATER BRIDGE, ACROSS THE DEE, ABERDEENSHIRE.

Plate 1.

The centering here shown is that used for the middle arch of Ballater Bridge. This bridge occupies the place of a former structure which was swept away by a flood in 1799. The present bridge was erected by the late Mr. Telford, and opened for the passage of carriages in the year 1809.

The bridge is situate on the Dee, and is distant 40 miles from Aberdeen: it has five arches, the centre one being 60 feet in span. The bridge is of granite, and the contract price was £3300, but the work is said to have entailed a heavy loss upon the contractor.

As a design, this centering presents no peculiarity worthy of notice, except that in order to guard against damage from ice, and to avoid obstructing any floating object in the river, which at this place has a very rapid current, the main horizontal beams of the centering are elevated 6 feet above the springing of the arch. The surface of the water in this, as in most of Mr. Telford's bridges, nearly coincides with the level of the springing, so that the timber of the centering thus disposed would be out of reach of the water.

The principal parts of this centering are the two main rafters extending from the springings of the arch to the top of the king-post; the vertical king-post into which the two rafters are framed and to which they are strongly secured by bolts passing through an iron plate with radiating arms; and the main tie beam secured to the king-post by an iron strap, and bolted to the rafters at the crossing. From the foot of the king-post proceed two struts inclined to the horizon at an angle of 40°. These struts are framed into the curved timbers which support the covering boards. Two additional struts are framed into the rafters on each side of the arch between the principal rafter and the springing.

At a height of 7 feet above the lower or main tie beam is another tie beam or collar beam, which extends across the centering, and from the points where this beam crosses the king-post spring two short struts inclined at nearly the same angle as the main struts before mentioned. From the extremities of the upper tie beam, two rafters, about 26 feet in length, extend to the upper course of footings of the pier, where the projection of the stone-work affords them a shoulder to abut against. To give further stiffness to the framing, two longer rafters, 37 feet in length, extend from the extremity of the short struts last described to the same point of support on the sides of the pier as the last-mentioned rafters rest upon. All the timbers are bolted to each other at the crossings.

We have to regret that the details at our command respecting this centering are somewhat scanty. In addition to the elevation with which the Plate presents us, it would have been advisable to have either a cross section of the timbers, or at least to have in figures the scantlings of the several timbers. In the absence, however, of this information it may be conjectured that the centering was constructed of fir timber, and the dimensions of the principal parts were 12 inches in depth, as shown in the Plate, by about 7 inches in breadth.

Although it might be objected to this centering that the timbers are not disposed in a very scientific fashion, it should be observed that the whole may very conveniently be taken to pieces when it has served its purpose in one arch, and reconstructed of a smaller size without cutting much of the material to waste. This is often of great importance in the practice of bridge building, and it probably caused in the present example the employment of a large quantity of timber of small scantling, and all of nearly the same scantling, in place of a more scientific arrangement, by which ulterior views might not have been so well answered.

AMERICAN TIMBER BRIDGE.

Plates 2, 3, and 4.

These Plates exhibit the construction of a peculiar kind of truss bridge which was patented in 1835 by Ithiel Town, an American engineer, and which has since been extensively adopted for some very large works in that country.

The form of this truss will be seen from the elevations in Plate 2, which are applicable to a double truss bridge on Mr. Town's principle, there

being a double truss on each side of the road-way. Each truss consists of a series of diagonal or truss braces inclined at an angle somewhat steeper than 45°, crossed by another series inclined at the same angle in an opposite direction, and of a horizontal string piece on each side at top and bottom, the whole firmly secured by wooden trenails of hard wood, as shown in Plate 2. The height of the truss is usually about one-tenth or one-twelfth the span of the bridge or distance between the piers. The plan in Plate 2 shows the way in which the double trusses are put together with string pieces between them, and the transverse section shows one end of a pier with the double truss erected on it and part of the roof extending across from one double truss to the other.

There are two very distinct methods of building these bridges, according as the road-way is placed at the top of the trusses or on the level of the lower string pieces. The first section in Plate 3 shows the first construction, and the second shows the road-way at the bottom with the whole height of head-room confined to the depth of the trusses by the beam which rests on them at the top. The first construction is that which will commonly be advisable when the span is not great, and when, consequently, it would be unnecessary to employ trusses of so great a height as 15 feet, which is about the minimum of head-room required for railways. In the case of wide roads also which are not railways, and where, consequently, suspension posts in the centre would be inadmissible, the first construction will be invariably necessary, however wide the span, unless additional strength be given to the floor beams.

The second mode of construction has the advantage of enclosing the road-way both by a side and top covering or roof, and in both the designs exhibited in Plate 3 a wooden boarding to protect the trusses from the weather is shown on each side of the bridge.

Plate 4 is a plan showing the horizontal beams and braces above the trusses in a bridge of the second construction.

The writing and figures on Plate 2 describe the names and scantlings of the several timbers seen in the cross section, in addition to which it may be observed that all those horizontal parts of a truss which run the whole length of it, and are secured to the truss braces by trenails, are called string pieces, those at top and bottom of the truss being respectively termed top and bottom string pieces, while those between the top and bottom are called intermediate string pieces. One intermediate string piece of a single plank is shown on the inside of the truss in the lower section in Plate 3. The centre or middle string pieces are those which

run between the two series of truss braces, and are here shown (Plate 3) as composed of square timber, but they are commonly of two single planks similar to the string pieces marked B in the Plate.

The form, dimensions, and position of the truss braces will be fully understood from Plate 2.

The floor beams shown in the two sections of Plate 3 are transverse timbers which support the longitudinal floor joists. Their depth should in all cases be at least twice their breadth.

The floor joists are those of which the ends are seen in the sections of Plate 3 resting on the floor beams. They are usually 4½ to 6 inches square, and they extend the whole length of the bridge in order to support the floor planks.

The floor planks are of various widths, from 6 to 12 inches, and from 2½ to 4 inches thick. When only one thickness of plank is used they are laid at right angles across the bridge, and when two are used they are laid obliquely to the line of road, so as to cross each other nearly at right angles.

The horizontal braces at the top shown in Plate 4 are either framed in diagonally between the top beams, or consist of long planks of suitable dimensions, spiked and trenailed to the top of the beams, so as to give the most secure support, and to keep the trusses in a straight line at top. Mr. Town considers this a very important support, and recommends it to be effected in the most secure manner. The form of these braces shown in Plate 4 is applicable to a bridge having its road-way between the trusses, the floor beams here resting on the top of the trusses, and the braces being framed diagonally between the beams.

Principal rafters shown in Plate 3 are of 10 or 12 feet in depth by 4 or 5 inches thick, according to their length. They are firmly secured to the floor beams at their feet.

Side braces (see lower section, Plate 3) are pieces about 4½ × 5½ inches, connected with the side truss and secured to the floor beam and principal rafter. They serve to confine the side trusses in a vertical position, and to prevent them from swaying or leaning to either side. The braces in the upper section of Plate 3 extend diagonally from the ends of the tie beam to the centre of the floor beam, as shown. They serve the same purpose as the side braces in the lower section.

The trenails used for securing the truss braces and string pieces together, as shown in Plate 2, should be of white oak or other hard wood, and in the large bridges of America are usually 2 inches in diameter. They should

be exactly fitted to the augers used for boring the holes, so that when seasoned they may drive tight and make solid work. The patentee observes that the trenails may be made different ways, but the best and most economical is to saw them out square from plank, with a circular saw, and then turn them with a small lathe, attached to some water or other machinery. They should be unseasoned, to be easily made, but must afterwards be well seasoned before driven into the work; they will season quick, or may be kiln-dried. Tallow or oil, &c., may be rubbed on them, to make them drive more easily, if necessary.

We are not aware of any situation in this country or in Europe where a bridge on Mr. Town's principle has been erected. In America, however, there are already many of these bridges, on a scale of magnitude truly gigantic. Among the most important of these is the one erected by the celebrated engineer Moncure Robinson, Esq., for carrying the Richmond and Petersburgh Railway across the falls of James River at Richmond. The length of this bridge across the river is 2900 feet, and the trusses are supported on eighteen granite piers, the distances between the piers varying from 130 to 153 feet. The piers are founded on the granite rock over which the rapids flow. Their height above the surface of the water is 40 feet, and they are carried up with a batter of 1 inch in 2 feet vertical, up to this height of 40 feet above the water, where their dimensions at top are 4 feet in breadth by 18 in length. The masonry consists of regular courses of stone dressed on the beds and joints, but left rough or merely scabbled on the outside faces. The floor in this bridge is on the top of the truss frames, and the depth of these being 20 feet, the road-way is carried horizontally across the river at an elevation of 60 feet above the water. This bridge was completed in September, 1838, having occupied less than two years in its construction, its cost amounting to about £24,200 sterling. We doubt whether any part of the world could produce an instance of a work equal in magnitude to this being executed for so small a sum. We dare scarcely trust ourselves to guess what a bridge of the immense length of 2900 feet, or considerably more than half a mile, would have cost in the hands of one of our English engineers; assuredly, looking to examples which surround us in every direction, we might change the units of the American engineer into tens, and call the twenty-four thousand 240,000, without incurring the imputation of exaggerating.

In addition to this great work on Mr. Town's principle, executed by

Mr. Moncure Robinson, may be mentioned another on the same principle across the Susquehannah, 2200 feet in length, with spans of 220 feet; also the following,—at Nashua, in New Hampshire; Newburyport, Massachussetts; Springfield, Massachussetts; Northampton, Massachussetts; at Providence, Rhode Island; one near Philadelphia, in New Jersey; one across the Delaware, above Trenton; one 736 feet in length on the Harlem Railroad, near New York; four over the North and Mohawk rivers, near Troy; another over the Schuylkill, near Philadelphia, for the Baltimore Railway; two on the Railway from Petersburgh to Raleigh, Upper Canada; one at Tuscalossa, Alabama; one at Circleville, Ohio, &c.

These bridges may be constructed of any kind of timber, however soft, provided planks of about 27 feet in length can be sawed out of it. White pine, spruce, and poplar have been extensively used in America, but oak is objected to on account of its tendency to spring or warp, if not well seasoned.

Mr. Town's principle of lattice frames or trusses appears to possess some advantages over most other kinds of wooden bridges.

1. The lattice bridge may be constructed of any span up to 300 feet, with common planks, not exceeding 30 feet in length, and of course for smaller spans a shorter length of plank will be sufficient. The ease with which the timbers required for it may be transported affords a great advantage for military bridges, and in all cases where carriage is expensive.

2. It may be adapted with absolutely the same degree of ease and convenience to an oblique or skew crossing of a river or road as to one at right angles to its course.

3. There is no lateral pressure or thrust against the piers or abutments, the frames merely resting on their tops, so that a much less strength of masonry is required than in most other kinds of bridges.

4. The short lengths and small scantlings of timber required expose every part to view, so that any appearance of weakness or dry rot, or any other defect, can readily be discovered, and the plank rejected, whereas such imperfections would escape notice in the interior of a large log.

5. The effect produced by pressure on the bridge, by the shrinking of the timber, and by other causes to which it is subjected, consists of a strain of tension or compression only, which is immediately subdivided and distributed so as to be perfectly unproductive of injury to any particular part.

6. The bridge has a degree of stiffness and freedom from both vertical and lateral motion superior to most other works, whether in iron or wood;

and this advantage is possessed in connexion with the practicability of always securing a perfectly horizontal road-way.

7. The covering of bridges with a light boarding to protect them from the weather is an important consideration, the value of which is well understood in America. The frames of these bridges present a prepared surface for the covering boards, and where the road-way lies between the side trusses the roof rests upon the tops of the trusses without any additional weight of timber, except that required for the mere roof itself.

8. These bridges are put together in America with remarkable rapidity; as an instance of which may be mentioned a bridge on the railway from Philadelphia to Norristown, which consists of three openings of about 190 feet each. This bridge, constructed across the Wissahiccon at a height of 78 feet above the bed of the river, occupied no more than ten weeks.

LADYKIRK AND NORHAM BRIDGE, OVER THE RIVER TWEED.

JOHN BLACKMORE, ESQ., ENGINEER.

PLATE 5.

This bridge consists of two arches, each of 190 feet span with 17 feet of rise, and a pier between them 20 feet in thickness, making the whole distance between the abutments equal to 400 feet. The bridge is of timber with stone pier and abutments.

In the few words we shall have to say respecting this bridge we shall have to confine ourselves to generalities, because at present we are only in possession of the general plan and elevation shown in the Plate, without details of any kind; so that the detailed account of the structure, which has been promised by the engineer, must of necessity be delayed until the second edition of this work.

The general arrangement of the timber in the arches of this bridge is worthy of attention, as it presents a combination which is remarkably light and elegant, at the same time that it possesses considerable strength and stiffness. On reference to the elevation it will be seen that the lower part of the arch is composed of a series of planks 6 inches in thickness, laid close upon each other. The length of these planks, being about 18 feet, serves to divide the whole ring of the arch into thirteen compartments, in the first of which, commencing at the springing, there are eight thicknesses of plank, and the thickness diminishes towards the crown by one plank in each compartment, till at the crown itself, and in the compartment on each

side of the crown, there are only three thicknesses. The planks of each compartment are securely bolted together by two bolts in each. The upper part of the arch consists, like the lower, of a series of planks arranged also in a tapering form, but the greatest thickness is here given at the centre, where there are seven planks, and this thickness gradually diminishes till in the last compartments, namely, those at the springing, there is only a single plank. The braces which radiate to the centre of the lower part of the arch unite the planks of the upper and lower parts at the ends of each compartment, and the diagonal braces extend from the top of each radiating brace to the foot of the adjoining one. Such is the arrangement of one rib of the arch, from which it will be seen that the bridge is essentially one which resists by compression. Conceive a weight placed upon the crown of the arch, and the effect is immediately that of tending to compress each of the diagonal braces, and before any part of the lower rim of the arch can yield in the slightest degree, it is essential that the whole mass of timber composing it shall be forcibly compressed to a considerable extent. The arrangement of the whole is so well adapted to distribute and convey to the strongest part, namely, the springing, any pressure which may come upon any part of the structure, that we cannot fail to pronounce the design to be one of great merit and ingenuity.

This form of bridge, although differing very widely from Mr. Town's principle, which has been so extensively acted upon in America, possesses in common with his the advantage of requiring only timber of small dimensions. This is a great recommendation in every district where timber has to be brought from a distance and where freight is expensive. In the bridge before us the greatest length of any single piece of timber is about 28 feet, this being the length of the longest diagonal braces, and the great majority of the pieces do not exceed 18 feet.

The road-way over Ladykirk Bridge having only the narrow width of 18 feet, no more than two ribs are necessary, as the road-way girders are not required to be of unreasonable dimensions when the bearing is no greater than 18 or 20 feet. It is obvious, however, that on account of the great strength required in girders having a greater bearing than this, the employment of this kind of bridge would not be judicious for very wide road-ways, unless a system of roofing and suspension from the centre could be introduced, similar to that which has been practised in many of Mr. Town's bridges. Again, as this divides the road-way into two separate lines, it is objectionable for any species of public road, but presents no disadvantage in the case of railways.

The details of fixing the road-way of this bridge with the framing beneath the girders must, for the reason already assigned, be for the present postponed until those full particulars are obtained which are necessary to the complete explanation of the work.

TIMBER BRIDGE OVER THE CLYDE AT GLASGOW.

Plates 6, 7, 8, and 9.

This is a bridge erected over the river Clyde in the years 1831 and 1832, between the Broomielaw Bridge and the old bridge of Glasgow; engineer, Robert Stevenson, Esq., of Edinburgh.

The bridge consists of fourteen arches, each of 34 feet span, measured from centre to centre of the piles forming the piers, giving in all a water-way of 476 feet, uninterrupted except by the narrow width of the piles or piers which support the structure. The road-way is 32 feet in width between the parapets.

Plate 6 shows the elevation and plan of one-half of the bridge, from which it will be seen that each arch consists of seven ribs, each composed of three beams, namely, two diagonal braces and a horizontal straining beam. The diagonal braces are 13 inches in depth by 12 in breadth, and the straining beams are 12 inches square.

The bridge is supported upon thirteen rows of bearing piles, seven in each row. These piles are set or pitched at the distance of 5 feet 2 inches apart, measuring from centre to centre, as shown in Plate 8. Each row of these piles composes what may be called a pier, as they form a substitute for what would otherwise be a solid pier of brick-work or masonry. At each end of each pier there is a shore or brace pile driven down at the distance of 13 feet from the outer bearing pile, as shown in Plate 8.

The piles which compose the piers are secured to each other by four pairs of beams called collar braces. These extend transversely across the line of the bridge on each side of the piers, and are of unequal lengths, as shown in Plate 8.

The first or uppermost pair of these braces is placed immediately under the road-way beams; the second pair is fixed 4 feet 3 inches under the uppermost, measuring from the upper edge of the one to the upper edge of the other, and forming the points of abutment for the diagonal braces hereafter described. The third pair of braces is fixed midway between the

second pair and the level of summer water, as shown in the section and elevations, and the fourth is placed at the level of summer water.

Plates 6 and 7 show the ends of these collar braces with the screw bolts and nuts which secure them to the piers. The first and second pair are secured with bolts of 1 inch square, and the third and fourth with bolts of $\frac{3}{4}$ of an inch square: the fourth pair of braces are however secured at their extremities to the heads of the shore or brace piles by two bolts of 1 inch square. Plate 8 shows that the fourth pair of braces extend beyond the pier from one brace pile to the other, and that the intermediate collar braces also extend beyond the piers till they meet the cutwater and tailwater braces, which will be presently described, and to which they are secured by bolts similar to those which fasten them to the piles.

The brace pile on the up-stream side of the bridge is called the cutwater brace pile, and that on the lower side is the tailwater brace pile. The sloping braces shown in fig. 8, which abut on the heads of these piles, and are fitted under the ends of the first pair of collar braces, are called respectively the cutwater and tailwater braces. These braces are fixed with two screw bolts of 1 inch square to the cast iron shoe on the brace pile head, and are secured at top by a similar bolt passing through the top of the outer bearing pile of the pier. The cast iron shoes for the bottom of the braces are somewhat in the form of the letter H, the two side plates being about 18 inches long, and formed to the proper angle. The cross bar is 12 inches long, and the whole about 10 inches in breadth, and averaging $\frac{5}{8}$ths in thickness. The shoes are fixed with two screw bolts, of 1 inch square, passing through the head of the piles and the lower tails of the shoes, and one $\frac{3}{4}$-inch rag bolt, 8 inches in length, driven into the top of the pile. The shoes for the top of the braces are of the form of the letter L inverted, the upright plate being 12 inches square, and the upper or horizontal plate 18 inches × 7 inches: the thickness on an average is $\frac{5}{8}$ths of an inch and the shoe is fixed to the piles and collar braces with four $\frac{3}{8}$-inch rag bolts in each shoe with countersunk heads. The cutwater braces, or those at the up-stream end of the piers, are faced with a cast iron sheath of one-half inch in thickness, in order to protect the bridge from injury by ice and other matters which may come against it.

The diagonal braces forming the arches spring from the upper edge of the second pair of collar braces, and those at the extremities of the bridge spring from the projecting course of masonry formed for that

purpose on the abutments. The length of the scantlings and position of these diagonal braces will be seen from the elevations in Plates 6 and 7, and the form of the shoes in which they rest will appear from these elevations and from the transverse section, Plate 8. Each shoe is furnished with a bracket, and the foot of each brace is notched to receive this bracket, which, thus fitting into it, prevents the brace from shifting.

The straining beams shown in the two elevations rest between the ends of the diagonal braces last described. A plate or sheet of malleable iron is introduced at each of the joints between the braces and the straining beams, to prevent the fibres of the one from being compressed into the other. The superficial area of these malleable iron plates is equal to the area of the joints, and at the top they overlap the upper end of the diagonals to the extent of 4 inches, and in a similar manner on the underside they lap under the straining beams to the extent of 3 inches. The plates are fixed to the timbers with three spikes, 4 inches long, in each of the over and underlapping sides.

The road-way beams or joists shown in the plan (Plate 6) are laid in seven equidistant parallel lines on the top of the pile heads, and rest on the upper edges of the straining beams. These beams are joined on each pier by a vertical scarf of 16 inches in length; each joining being fixed to the pile head by two oak trenails of $1\frac{1}{4}$ inch in diameter, and secured longitudinally by two straight straps of malleable iron, 2 feet in length, and 3 inches in breadth, by $\frac{1}{2}$ inch in thickness, placed one on each side of the beams, and secured thereto with two $\frac{3}{4}$-inch bolts. Where the road-way joists rest upon the straining beams, they are bolted to the latter with screw bolts $\frac{7}{8}$ inch square, one bolt at each end and one in the middle of the straining beam.

The planking of the road-way shown in Plate 6 is fixed with two spikes at the crossing of each of the road-way joists, and projects 6 inches over the face of the outside joists.

Plate 9 is a section on a large scale, showing the foot-path, the curb beam which separates the road-metal from the foot-path, and the ends of the road-way joists and straining beams. The curb beams are secured to the planking and to the beams below by $\frac{7}{8}$-inch screw bolts, of 29 inches in length, one of which is placed at every 6 feet, and an additional bolt at every joining. On the top of the wooden curb is laid a line of cast iron guard plate of the curved form shown in the section: this is secured to the curb beams by bolts.

The whole of the timber for the bridge was directed to be of good sound

Memel or of the best Quebec red pine. The timbers within the range of the tides are payed over with two coats of tar; and those above the level of high-water mark are planed and clean dressed, and finished with three coats of white lead oil paint.

The whole of the iron work was of the best Muirkirk or Lowmoor iron, and the whole of the bolts, shoes, sheathing, &c., were made according to patterns furnished by the engineer during the progress of the work.

The abutments were founded upon piling which it will be unnecessary to describe particularly, as it could convey little information unless the reader possessed an accurate knowledge of the strata on which they rest. The extent and form of the abutments and wing walls are also matters of mere local interest, as they depend on local circumstances of position and connexion which have nothing to do with the general design.

It may be mentioned, however, that the stone for the abutments, wings, parapets, &c., was procured from the same sites as for Mr. Stevenson's celebrated stone bridge over the Clyde, called Hutcheson Bridge, of which see the specification in the first volume. The stone parapet over the abutments is 18 inches in thickness and 4 feet in height, surmounted by a coping 20 inches in breadth by 8 inches in thickness. The parapet over the arches consists of a substantial line of post and rail, 4 feet 3 inches in height above the level of the planking.

WOODEN BRIDGE OVER THE CALDER AND HEBBLE NAVIGATION.

BY WILLIAM BULL, ESQ., CIVIL ENGINEER.

Plate 10.

This bridge was erected in the autumn of 1836 for carrying an occupation road over the canal.

The chord of the arc is 70 feet, the versed sine is 5 feet, and the width of the road-way 10 feet.

The arch is composed of four ribs of fir timber with cross braces. On the under side the timbers are wrought to the curve of the arch, and the ends abut on each other and radiate to the centre of the arch. The timbers are secured in their places by two wrought iron bolts passing through them in the line of the radii at each joint, and these bolts are connected at each end by iron straps let into the wood. The ribs are also secured to each other by wrought iron through bolts passing through

them and the cross braces in a horizontal direction. The through bolts have a strong cast iron washer at each end.

The road-way is composed of 3-inch deal plank, on which is laid a coating, ¾ of an inch in thickness, of pitch and tar mixed with small gravel, in order to protect the planking from the weather, and over this is placed a layer, 5 inches in thickness, of broken stone, to form the surface of the road-way.

The ribs are let into the springing about 6 inches at each end, wedged in with oak, and the chases run with lead.

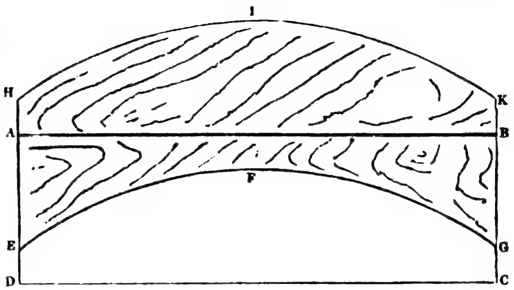
The abutments are formed of ashlar stone, unhewn except in the beds and joints, backed with rubble, laid in mortar, and grouted.

The wing walls consist of stone 5 inches in thickness, with 7 inches of bed, laid in mortar, backed with rubble, and grouted.

The foundations are sunk about 2 feet below the ordinary surface of the ground. The footings consist of a course of flat stones 5 inches in thickness, each stone having an area of 12 superficial feet.

The arch was formed and perfectly fitted in all its parts at the carpenters' yard belonging to the Navigation Company, and taken to the spot in separate pieces.

In this bridge a curved form is given to the timber of the arch without cutting any of the material to waste. To effect this, the curved part of each rib consists of two logs of timber, 18 inches square and about 18 feet in length. These logs are divided through the centre so as to form four half logs of 18 inches by 9 inches. The annexed figure will explain how the curved figure of the arch is cut out of the log without wasting any of the wood. A B C D represents a straight log which has to be cut into the form of an arch E F G. Let the arc E F G be described, and cut out the piece E F G C D, and place it above the log in the position of H I K B A. It is evident now that the whole figure H I K G F E has the curved form of the arch as required. It is true that the arc H I K is not precisely concentric with E F G, being in fact described with the same radius; but this does not in the least affect the strength of the arch, nor would it be possible to observe even in arcs subtending a much greater angle than those which are usual in



the practice of bridge building that these arcs were not strictly concentric. The straight pieces above the arch are 9 inches square in scantling, so that the four pieces required for each rib are cut out of a single log 18 feet in length and 18 inches square. The cross braces are small pieces of square timber, of which the dimensions are seen on the plan. The covering to receive the metalling of the road-way consists, as already described, of 3-inch planking. Two of these planks are cut without waste by dividing a 20-feet plank in the middle, and the whole bridge requires seventy-eight half planks of 10 feet in length, or thirty-nine whole planks of 20 feet, to form the road-way covering. The side boards to keep up the road-metalling are also of 3-inch plank set on edge, about seven 20-feet planks being required for this purpose for the two sides of the bridge.

NEWCASTLE, NORTH SHIELDS, AND TYNEMOUTH RAILWAY VIADUCT, ACROSS WILLINGTON DEAN.

Plates 11, 12, and 13.

This is a timber viaduct with stone piers, erected by Messrs. Green, of Newcastle upon Tyne, to carry the Newcastle, North Shields, and Tynemouth Railway across the valley of Willington Dean. Each extremity of the viaduct is joined by an earthen embankment, 33 feet in depth at one end and 36 feet at the other. The extreme length of the viaduct is 1048 feet, and its greatest depth from surface of ground to the level of rails is 82 feet. These dimensions will afford an idea of the heavy mass of embankment which is saved by the substitution of a viaduct in this place. The structure consists of seven arches, namely, two of 115 feet span, four of 120 feet, and one of 128 feet.

FOUNDATIONS.

A reference to the general elevation in Plates 11 and 12 will show that the bed of the river is composed of alluvium resting upon a more solid stratum of clay, and that the depth down to the clay is inconsiderable under the two end arches at each extremity of the viaduct, but that in the centre the thickness of the alluvial soil is much greater. As the stratum above the clay does not possess sufficient solidity for a foundation, it became necessary to drive piling down to the clay under the two centre piers; and as for the other piers and the abutments, the proximity of the clay to the natural bed of the valley afforded complete facility for founding directly upon it without the use of piling. The two abutments and all the

piers, except the two centre ones, are founded from 2 to 6 feet below the surface of the clay; one of the centre piers is founded on piling 36 feet in length, and the other on piling 32 feet in length. There are eighty piles in the foundation of each of these two piers, with sleepers and planking, as shown in the elevation and sections in Plate 13. All the piers and abutments have four courses of footings. The footings of the two centre piers are surrounded by a row of fender piles about 14 feet in length.

PIERS AND ABUTMENTS.

The two end piers are carried up to the springing in the square form shown by the plan in Plate 11: they have each a square projection at the ends, 2 feet 11 inches beyond the line of the arches and 7½ feet less in width than the thickness of the pier. These small piers are of solid masonry up to the springing, with the exception of two small hollows in the interior, which are filled with rubble, as shown in the plan. The dimensions of the four other piers will be seen from the elevation and the transverse sections. It will be observed that each of these has a bevelled set-off all round it, at about one-third of the height from the footings to the springing, and that above this set-off the length and breadth are each diminished 1½ feet. The ends of the piers are carried up perpendicular, but the sides have a batter both above and below the set-off of 1 in 24. The general thickness of all the piers at the under side of the springing is 15 feet by a length of 22 feet. The large piers, like the smaller ones, have each a projection at the ends. This projection extends 2 feet 11 inches beyond the general line or face of the bridge, and is 10 feet in width below the set-off, and 8 feet 6 inches above it. At about half-way between the set-off and the springing course there is a Gothic moulding, above which the projection is diminished so as to be only 1 foot 11 inches beyond the general face, as shown in the transverse sections in Plates 11 and 13. These piers are composed of solid masonry, with the exception of two hollows in each of 6 feet by 4 feet, which are filled with rubble, as shown in the transverse and longitudinal sections in Plate 13. The abutments and wing walls at each end of the bridge form a solid mass 60 feet in length by 25 in breadth, carried up with four hollows of 20 feet by 5 feet. The abutments have each a projection at the ends, corresponding with those of the piers already described. The piers and abutments above the springing course are continued in the form of pilasters up to the string course, between which and the springing they have each a Gothic moulding similar to that below the springing in the large piers. Between the springing and this upper moulding the width of

all the pilasters is 7 feet 10 inches, and above the moulding their width is diminished to 6 feet 6 inches. The length also diminishes as shown in the various transverse sections in Plates 11 and 13.

ARCHES.

These are of a very peculiar structure, and present almost the first examples in this country of that laminated form which was successfully adopted in the Pont d'Ivry across the Seine. The arches of this viaduct are built of 3-inch deal planks from 20 to 45 feet in length, and 11 inches in breadth. Each rib contains two planks in width, and is regularly built in courses in such a way that each alternate course consists of two whole deals and of one whole and two half deals. The planks also break joint longitudinally, and the whole are firmly secured by oak trenails, each of which passes through not less than three of the planks in thickness. Between each of the courses is placed a layer of brown paper saturated with boiling tar. Each rib is 1 foot 11 inches in breadth, as seen in the transverse section, Plate 11, and is composed of 15 courses of 3-inch plank. Each pier has six cast iron socket plates fixed to it to receive the ends of the ribs. These plates are secured to the pier by four iron bolts in each, as shown by the plan and longitudinal section in Plate 13. The ribs are tied to each other transversely by wrought iron bolts and diagonal braces, as seen in the plans, Plates 11 and 13, and in the transverse section, Plate 11.

SPANDRILS.

The spandrils are of open timber-work, and consist of main dividing beams with a series of diagonal and vertical struts. These dividing beams are called spandril beams in Plate 12. They are long pieces of timber 14 inches square, and are placed in an oblique direction from the crown of the arch to the moulding already described on the pilasters between the springing and the string course. The radiating struts are those between the rib and the dividing beam, and the vertical struts extend from the dividing beam to the longitudinal timbers of the road-way. The arrangement of the timbers in the spandrils with the bolts which secure the principal parts to the ribs and to each other will be seen on reference to the several elevations given in these Plates. All the ribs have spandrils of the same general construction.

ROAD-WAY.

The longitudinal beams of the road-way are 14 inches square. They are laid in three parallel lines passing over the crown of each rib. The breadth from outside to outside of these beams is 18 feet. The transverse girders

are 12 inches in depth by 6 inches, and 23 feet 2 inches in length, so that they project on each side something more than 2½ feet beyond the road-way beams. These girders are placed 4 feet apart. The planking, 3 inches in depth, is laid longitudinally 23 feet 8 inches in width, and the railway sleepers are laid directly upon the girders. On each side of the road-way between the two abutments is a substantial wooden railing, five bars in height, with posts 7½ feet apart. The wing walls, at each end 60 feet in length from the face of the abutments, are surmounted by a stone parapet with small pilasters at the ends.

VIADUCT ACROSS OUSE BURN DEAN.

Plates 14 and 15.

This is another work of Messrs. Green, also on the short line of the Newcastle, North Shields, and Tynemouth Railway. It consists of five wooden arches of 114 and 116 feet span, and four stone arches, namely, two at one end of 43 feet span each and two at the other end of 36 feet span each. The necessity for these smaller arches at the end arose in consequence of the irregular shape of the valley, which would not permit the introduction of more than five arches having the same considerable rise as those in the centre of the depression. It will be unnecessary to go through a particular description of this viaduct, as its construction closely resembles that of Willington Dean, which forms the subject of the last description. There is some difference in the foundations of the two works, which the plans will sufficiently explain. The road-way also is something wider in the Ouse Burn Viaduct, being here 26 feet wide between the rails, and having besides a foot-path on one side 5 feet in width. Each viaduct, however, contains the same number of ribs, and the general construction of the two works is precisely similar.

UPPER WOODEN BRIDGE AT ELYSVILLE, OVER THE PATAPSCO RIVER, ON THE BALTIMORE AND OHIO RAILROAD,

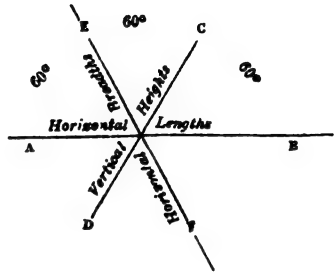
EIGHTEEN MILES FROM BALTIMORE.

Plates 16, 17, and 18.

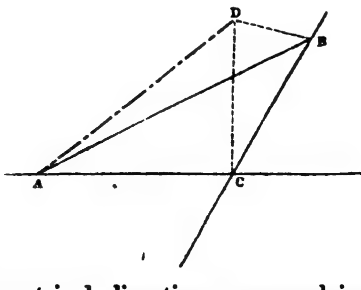
This bridge consists of two arches, each of 150 feet span.

Fig. 1, Plate 16, is an isometrical representation of one-half of the western arch next to the pier, showing the manner of framing and the whole construction in all its details, also the cast iron skew backs for the

abutment of the arch braces over the pier. The pier in this bridge is built at an oblique angle to the direction of the railway, as seen in figs. 7 and 9, but in the isometrical representation the pier is shown at a right angle to the direction of the railway, and consequently the floor beams over the pier, and the corresponding suspension posts of both ribs, are also at right angles opposite to each other.



Although this drawing may appear confused at first sight, a little examination will render it intelligible. It will be seen that all the horizontal lines in one direction, that is, lengthwise of the railway, lie parallel to the edges of the plate; and all those which represent horizontal breadths and vertical heights are inclined at angles of 60° to the horizontal lengths, as shown by the annexed figure. Now dimensions may be taken from the drawing in any of the directions indicated by these three lines, and if it be required to measure a part of the structure which does not conform to either of these directions, it may readily be reduced to its true length by the rules of isometrical projection. In the drawing before us the floor beams and the horizontal rafters from which the floor beams are suspended have their lengths in a direction coinciding with A B in fig. 1, their depths coinciding with C D, and their breadths with E F. The transverse beams which lie in a direction at right angles to the line of the railway, have their lengths coinciding with E F, their depths with C D, and their breadths with A B. Also the vertical suspension posts have their lengths coinciding with C D, and their other dimensions coinciding with A B and E F respectively. The radiating arch braces not being either horizontal or vertical, but inclined to the horizon, cannot be measured from the drawing without a reduction similar to that in the annexed example (fig. 2). Let A B be one of these braces meeting the horizontal and vertical lines respectively in the points A and B. To find its true length, draw C D equal to C B, and perpendicular to A C; then A D is the true length of the radiating brace which appears in the drawing in a direction coinciding with A B. The scantlings of the braces may be measured in the isometrical directions, as explained



for the horizontal beams from the figures 3 to 6 in Plate 16; and by means similar to those pointed out for lines in different directions, the dimensions of every part of the framing may be obtained from this drawing.

Fig. 2 is an isometrical drawing of the cast iron skew back for the arch braces over the abutments.

Fig. 3.—Isometrical section of main arch brace,
4. " " 2nd ditto,
5. " " 3rd ditto,
6. " " 4th ditto,
showing their construction and exact dimensions on an enlarged scale.

Fig. 7, Plate 18.—Plan of the floor of the western arch of the bridge, with pier and abutment, showing the general arrangement of the floor beams and their braces and the relative position to the suspension posts of both ribs.

Fig. 8, Plate 18.—An elevation of the western arch, corresponding with fig. 7.

Fig. 9, Plate 17.—General plan, showing the position of the pier, the two abutments, and the wing walls.

Fig. 10, Plate 17.—General elevation of the bridge, as finished, showing the weather boarding and the roof, with an oblique front view of the eastern entrance.

This bridge was erected in the years 1838-9, to carry the Baltimore and Ohio Railway over the Patapsco river. At the same time another bridge was erected on the same principle, a quarter of a mile lower down the stream, this double crossing being necessary to cut off a large and abrupt bend of the river. The Plates above described represent the upper of the two bridges. The lower bridge consisted of three arches, each of 110 feet span, the united length of the two bridges being 675 feet. The cost of the two structures for masonry was \$10,928, and for the wood-work \$17,653, making together \$28,581. The lower bridge has its floor supported at one-third of the depth of the truss frames from their top, while the floor of the upper bridge rests as shown on the lower chords, and immediately on the top of the pier and abutments. The lower bridge is weather-boarded on the outside of each rib for the whole depth, and on the inside down to the floor, which last acts as a roof; and the top of each rib is covered with sheet zinc, nailed upon sheeting boards, supported on little rafters. The upper bridge is weather-boarded on the sides and roofed with shingles; and the timbers of the roof are protected from being fired on the inside by a ceiling of sheet iron.

These bridges, in one of the main features of their framing, namely, the radiating arch braces, resemble the celebrated bridge over the Rhine at Schaffhausen. With this principle, however, is combined a system of struts and tie rods which give to the present bridge a stiffness and freedom from vibratory motion which that famous structure could not boast of. It is said that even the trotting of a dog over the bridge of Schaffhausen causes it to tremble.

The Elysville Bridge is built of the American white pine which has been used for most of the bridges throughout the country. In determining the dimensions of the several timbers and their abutting joints, great care was taken to secure ample section and surface to prevent the crippling or bending of the timber by the extreme pressures to which it would be subjected.

This bridge affords a fine example of the employment of wood in the formation of a level road-way across a great span, at a cost too which, compared with that of bridges in this country, appears almost incredibly small, even taking into account the cheapness of timber in America. The attention of the reader is directed to the manner in which the upper chords or straining timbers are connected by opposite keys on the outside,—to the mode in which the lower chords are held together by block keys between them,—to the attachments of the cast iron skew backs of the arch braces to the lower chords,—to the way in which the main arch brace is built of three timbers, and the other principal braces of two timbers each,—and to the general separation of the timbers wherever it was practicable by an interval sufficient to permit the free access of air to all their sides;—to the use of cast iron plates between all the principal abutting surfaces,—to the mode in which the floor is supported immediately *at* the suspension posts, thus avoiding cross strains upon the chords between the posts,—to the way in which the longitudinal flooring joists, which lie immediately under the rails, are formed of two pieces bolted together and breaking joint on the alternate cross floor beams,—to the perfect facility for adjusting all parts of the framing by means of keys and screws,—and, finally, to that general effect of the combination of all the parts of the structure which tends to distribute over the whole framing the partial and shifting pressures which passing loads bring upon it, and to give the greatest strength and stiffness with a limited amount of material.

The design of this bridge was made in the summer of 1838, by Benjamin H. Latrobe, principal engineer of location and construction in the service of the Baltimore and Ohio Railroad Company, and the erection of the

bridge was superintended by James Murray, assistant engineer, to whom is due the credit of having suggested the manner in which the main arch braces should be built of three pieces, as described above. The first locomotive engine which passed over the upper bridge weighed with its train about 70 tons; this load occupied successively the length of each arch (150 feet), and caused a depression while passing of not more than ¾ of an inch at the middle part of the truss. Since that time both bridges have borne, without yielding in any part, the daily action of a very heavy traffic, passing at speeds of from 10 to 20 miles an hour.

OLD AND NEW LONDON BRIDGES.

Plate 19.

(See Hosking's Practical Treatise.)

WESTMINSTER BRIDGE.

Plates 20, 21, 22, 23, and 24.

In the year 1735 the inhabitants of Westminster petitioned parliament on the subject of a bridge across the river for the convenience of their city; and although considerable opposition was raised against the measure, an act was passed to authorize a lottery, from the profits of which a bridge was to be erected from New Palace Yard to the opposite shore of the river. From some cause or other the first lottery which was attempted proved a failure, and a second act of parliament was passed in the year 1736, authorizing the issue of seventy thousand tickets of £10 each. It was enacted that 14 per cent. should be the profit reserved from the lottery, and this sum, amounting to £98,000, was to be applied in defraying the expenses of the lottery and in the erection of the bridge. For the latter purpose it is said that little more than £70,000 remained. The commissioners appointed by the act of parliament were empowered to build the bridge of any materials which they might think proper; and numerous designs were consequently sent in, amongst which were the two exhibited in Plates 20 and 21.

The design for a wooden bridge of thirteen arches to be built upon stone piers appears to have been completely matured at the time when Maitland's London[1] was published. He speaks of the design as one which

[1] The History of London from its foundation by the Romans to the present time; by William Maitland, F.R.S. London, fol. 1739.

was fully decided upon, and after describing the method proposed for founding the piers in caissons, continues, "upon these, and the two end piers, 'tis said will be erected thirteen wooden arches! which will not only greatly redound to the dishonour of the nobility, gentry, &c., in these parts, for whose convenience it is chiefly intended, but likewise to the kingdom in general, to have a disgraceful wooden bridge erected so near its capital city, which, as already has been shown, has not, nor never had, its equal upon earth, for opulency and number of inhabitants! Besides, the incomparable River Thames for the spaciousness of its stream, and depth of its channel, deserves one of the best and most pompous edifices to be erected over it. And for the avoiding reproach to the nation (for nothing looks more mean and beggarly than a wooden bridge) 'twere much better to be without a convenience at this place, than to have one of various materials."

Although Mr. Maitland's abhorrence of wooden bridges here carries him rather too far, it is probable that remonstrances of this kind contributed to the abandonment of a design which would certainly have been far inferior to the present structure.

The original project for erecting a bridge at Westminster adds another to the long catalogue of instances where popular prejudice has blindly and foolishly opposed almost every kind of improvement, both local and general, which has ever been introduced into the world. We are informed that the citizens of London, the inhabitants of the Borough of Southwark, the Company of Watermen, and the west country bargemen, severally petitioned parliament against the erection of the bridge.

The plan and elevation in Plate 20 exhibit a design for a wooden bridge, by Mr. John Wesley, of Leicester, who is styled on the Plate the father of John Wesley, founder of the Wesleyan Methodists. There is, however, some error in this statement, because Samuel Wesley was the father of the John Wesley from whom the Wesleyans take their name, and his residence was at Epworth in Lincolnshire, of which place he held the living.

His design for a bridge at Westminster, far from possessing any merit, appears to be a very puerile conception, and except for the celebrity of the name of Wesley, would probably never have attracted the least notice from amongst a great number of others which are still preserved as curiosities by Mr. Swinburne, the resident engineer at the bridge.

Plate 21 exhibits the design of Mr. James King for a wooden bridge with stone piers. This design was the one actually selected by the

commissioners, and according to which the bridge was originally built up to springing height. It is this design of which Mr. Maitland speaks so contemptuously in the passage above quoted. Whether owing to remonstrances similar to those of Mr. Maitland, or to some other cause which it might now be difficult to explain, it is certain that after the contract had been entered into for a bridge according to King's design, and after the whole of the piers had actually been carried up, the idea of a wooden superstructure was abandoned in favour of Labelye's stone arches. Mr. James King, whose design was thus superseded by that of Labelye, appears to have been an ingenious and able master of carpentry, and his name is frequently mentioned in very favourable terms throughout Labelye's reports. His wooden arches have an extremely antiquated appearance in the eyes of a modern engineer; but although the arrangement of the timbers is complicated, there is certainly more science and ingenuity displayed than in the other design shown in the preceding Plate.

It should be understood, however, that although the design in Plate 21 is commonly called King's design, yet it is only the wooden superstructure which can fairly be assigned to him. The openings of the several arches and the dimensions of the piers were all fixed by Labelye, who was intrusted with the whole management of the foundations and with the work of carrying up the piers to the proper height for receiving the wooden superstructure.

At the time the contract was entered into, in June, 1738, for the erection of the bridge according to the combined designs of King and Labelye, it had been decided by the commissioners that the structure should consist of thirteen arches, the piers for which should be of Portland stone. That these piers should be carried up of uniform breadth to the height of 1 foot above low-water mark. That over each of the piers should be carried up shafts of solid stone 44 feet in length and 15 feet in height, the breadth of these shafts to be 8 feet for the first 5 feet in height, 7 feet for the next 5 feet, and 6 feet for the upper 5 feet. Each shaft was to be surmounted by a torus or capping course, the under side of which corresponded with the level of high water. During all the time that he was employed in erecting the piers and shafts to support King's superstructure, Labelye appears to have been strongly impressed with the superiority of his own design for stone arches; and in his reports, and in the pamphlet which he published in 1739, during the building of the piers, he omits no opportunity of alluding to the facility with which

stone arches could be adapted to the original piers and shafts. It is a circumstance extremely complimentary to the genius and acquirements of Labelye, that he was able by his own clear and forcible arguments and explanations to induce the commissioners in the end to adopt his entire design of stone for the whole bridge. The engineering skill and sagacity displayed by Labelye, and the bold and calm confidence with which he contended against severe opposition in defence of his darling project, demand an additional share of admiration when we consider that he was by birth a native of Switzerland, that up to his twentieth year he had never heard one syllable of English spoken, that he lived in a time when the acquirement of foreign languages was difficult and rare, and, above all, when foreigners were not so universally patronized in England as in these days of toleration and liberality. By reference to Plate 22 it will be seen how the semicircular stone arches proposed by Labelye were afterwards sprung from the broad part of the piers, and how the shafts which had been carried up above now form part of the filling in between the arches.

Plate 23 shows the centering designed by King for the stone arches of the present bridge, and here undoubtedly there is much credit due to him as the constructor of an admirable piece of carpentry, scarcely inferior in arrangement to the best centering which modern ingenuity and skill have produced.

The whole length of the present bridge, as shown in Plate 24, is 1164 feet, of which the thirteen arches have an opening at the springing of 820 feet, the twelve piers have a section of 174 feet, and the remaining length of 85 feet at each end of the bridge is the distance from the springing of the first arch to the extremity of the wing walls. The entire breadth across the bridge is 44 feet, of which 27 feet are occupied by the carriage-way, and 7 feet on each side by the foot-paths. The span of the smallest arch at each end of the bridge is 52 feet, and the spans increase regularly by 4 feet in each arch throughout the six on each side of the centre arch, which last has a span of 76 feet. Besides the thirteen arches which afford the water-way for the river, there is in each abutment an arch of 25 feet span which is entirely dry at low water. All the arches are semicircular, and the whole are sprung from a line which is about 1 foot higher than ordinary low water. The gradual diminution of the span, and consequently of the rise of the arches from the centre to the ends of the bridge, affords a sloped road-way on each side with an inclination of 1 in 28. The centre piers, that is, the piers of the centre

arch, are each 17 feet in thickness at the springing, and the thickness of each successive pier is diminished by 1 foot, the first pier at each end of the bridge being 12 feet in thickness. The rule which Labelye appears to have followed in proportioning the thickness of his piers to the spans of the arches is the following: calling S the span of any arch, the thickness of its corresponding pier will be $\frac{S}{4}-1$. Thus, for example, the span of the fourth arch from either end of the bridge is 64 feet, and $\frac{64}{4}-1=15$ feet the thickness of the fourth pier; and the same of every other arch and its pier.

It is well known that in founding the piers of this bridge M. Labelye adopted the bold, and at that time novel experiment, of building above water within a caisson or chest of timber, which was finally sunk in the proper position to form a pier of the bridge. Of the two systems of founding under water, the one by coffer-dam and the other by caisson, the former is by far the more ancient, and had been commonly practised long before Labelye's time in France, in Holland, and even in England. Labelye, however, preferred the caisson system, on account of the great rise of tide in the Thames, the difficulty of keeping the water from oozing in at the bottom of the dam, and the expense which must have been incurred in the construction of so many separate dams as would here have been required. In preference, therefore, to a mode of proceeding in which he foresaw so many difficulties, Labelye adopted the expedient of founding by caisson. His ideas on this subject appear to have been entirely original, and the whole credit of invention appears to belong to him, if we except the rude form of caisson or chest which had been used in Holland in the construction of moles, dykes, sea embankments, and similar works. The practice had been to fill rough wooden chests with rubble stones, and sink them in the line of the intended bank or rampart, and then to fill in between them with earth and stones. An example of this kind on a very gigantic scale is exhibited in the works carried on by Napoleon at Cherbourg, where immense caissons were sunk in the construction of the breakwater. Before Labelye's time, however, the chests employed had been very small, probably not more than 16 feet square, and no care being ever taken to provide a bed for them, they had been allowed to sink and settle down in the accidental position which they first assumed. The caissons for Westminster Bridge, on the other hand, were about 80 feet in length and 30 feet in

breadth. They had to support an immense mass of masonry regularly built inside them, and had to be lowered down to a prepared bed, and repeatedly raised and lowered till they were found to rest firmly and securely. Under these circumstances much greater caution and many more expedients were necessary than in the management of the old caissons.

The construction of the caisson for building one of the large centre piers having been described by Labelye himself, it may be interesting to give the details, which will serve, with little variation, for those of all the other piers. The sides, he tells us, were at first intended to be 30 feet in height, but he afterwards found that 16 feet was sufficient. The bottom of the caisson was a strong grating of timber planked underneath. The sides were made of fir timbers 12 inches thick at bottom, and 9 inches at top. These timbers were planked over inside and out by vertical planking 3 inches thick, so that the whole thickness of the sides was 18 inches at bottom and 15 inches at top. The timbers were laid horizontally, pinned with oak trenails, framed together at the corners, and secured by iron work at the salient angles. These sides, when finished, were fastened to the bottom or grating by twenty-eight pieces of timber on the outside, and eighteen on the inside. These pieces of timber were 3 inches thick, 8 inches wide, and as long as the depth of the caisson. They were dovetailed at the lower end so as to fit into corresponding mortises cut in the kerb of the grating. They were secured in these mortises by iron wedges, and this system of fastening the sides to the bottom was adopted on account of the convenience it afforded for floating off the sides of the caisson, by simply driving out the wedges and striking the dovetailed timbers through the mortises in which they had been held. The sides of the caisson were prevented from being pressed in by the water outside by means of ground timbers wedged in between the sides of the caisson and the first course or plinth of the stone-work. The top of the sides was secured by beams laid across to support a floor which the masons and labourers used in hoisting stones out of the lighters into the caisson. As the stone-work was carried up also, struts and props were occasionally placed between the stone-work and the sides whenever such precautions appeared advisable. The bottom of the river for receiving the caisson was prepared by dredging out a space 6 feet in depth and 5 feet wider all round than the base of the caisson. The sides of this excavation were formed with a slope to prevent the earth falling in, and in order to guard against the silting in of loose matter by currents of the

river, a row of dwarf piles, with their heads about 4 feet above low-water mark, was driven at each end of the intended pier about 10 feet from the edge of the excavation. These piles were provided with grooves into which boards were fixed in order to form a close fence for the temporary purpose above described. When the excavation for the pier was supposed to be finished to the proper depth, every part of its surface was gauged in the most particular manner, in order to detect every inequality, and to be certain that the whole space was properly prepared to receive the caisson. The gauge employed consisted of a stone about 15 inches square and 3 inches thick, in the middle of which was fixed a wooden pole 18 feet in height. This pole was painted red, with divisions and figures in white, so that by noting the height which the water intercepted on the pole, the variation from a true level which any part of the bottom presented was at once most accurately ascertained.

One of the sides of the caisson had a sluice fixed in it near its bottom, so that by opening this the water could be admitted and the caisson sunk at any time required. When the whole was put together, all the joints were well caulked and the bottom and sides payed over with pitch. The first three courses of stone were placed in the caisson while it was kept floating; but the engineer adopted the precaution of twice sinking it, namely, on the completion of each of the two first courses, in order to see if it settled fairly on the bed prepared for it. When the caisson had been thus temporarily sunk, and when it was required to make it again float, the sluice was shut shortly before low water, that is, when the water in the river began to fall below the top of the caisson. The pumps were then set to work, and in about two hours' pumping the caisson was made to float.

When the third course of the pier had been set and cramped, the necessary observations were made for fixing the pier at right angles to the line of the bridge, and the caisson was sunk for the last time, the upper surface of the stone-work then reaching to within 2 feet of low-water mark. From that time till the completion of the pier up to high water, the building was carried on during the intervals which the tide afforded. As soon as the surface of water in the river was below the top of the caisson the sluice was shut and the water pumped out; the masons then continued to work till the tide again rose above the top of the caisson. At first it was necessary to leave the sluice open nearly till low water, in order to guard against the floating of the caisson after

it had been permanently fixed; but when a sufficient weight of masonry had been placed in it, this precaution was not necessary, and the whole time was available for building during which the water outside was below the top of the caisson, except the time occupied in pumping out the water low enough for the masons to work. Such, with little variation, was the method employed by Labelye in founding the two centre piers. It seems to have been an error on the part of the engineer to dispense with piling in the foundations of these piers, as considerable expense has been lately incurred in securing the foundations. About fifty-six piles were driven as fender piles to protect the two large piers while building. These piles were driven by the celebrated horse pile engine designed for the purpose by James Valouë, an ingenious watchmaker. The ram of this engine weighed 1700 lbs., and when worked with three horses, the ram falling from a height not exceeding 8 or 10 feet, gave about five strokes in two minutes. These fender piles were 34 feet in length, and 13 or 14 inches square. They were driven down 7 feet apart and 13 or 14 feet into the bed of the river.

We have already mentioned that the piers intended to support King's superstructure are concealed by the spandrils of the present bridge from the level of the low-water mark, that is, from the springing of the stone arches. These spandrils are built of Purbeck stone in radiating courses, as in the Rialto at Venice, the great arch at Vicenza, and in a groin arch which M. Labelye mentions at Blenheim. The spaces between the spandrils are filled in solid with ballast and rubble.

The piers, abutments, and arches are built of Portland stone. The arch-stones are mostly about 2 feet 4 inches in depth, and 2 feet 5 inches thick. The centre arch has forty-nine voussoirs or courses of arch-stones, and the number in each arch diminishes by two, the 52 feet arches having each thirty-seven voussoirs. The masonry of the piers is built solid, and below high-water mark is laid in mortar made with Dutch terras. The fronts of the piers and of the arch-stones, and the soffits of the arches, are chamfered. The spandril walls are surmounted by a bold projecting block cornice, which is carried round the projections of the piers which form recesses in the line of the foot-way. The centering, of which one of the ribs is exhibited in Plate 23, consisted of five fir ribs which rested on transverse and longitudinal oak sleepers, and the ribs were secured transversely to each other by cross ties. The timber used for the centering was mostly Riga fir, cut into logs 13 inches square. The quantity used for the centering

of the middle arch was about 160 loads.[2] The same centres which were constructed for the arches on the Middlesex side were afterwards used for the corresponding arches on the Surrey side.

Although upon the whole Labelye is entitled to great credit alike for the design and the execution of the works connected with Westminster Bridge, he certainly committed a serious error in leaving his caisson bottoms in such an exposed condition that one of them was actually damaged, and a considerable settlement occasioned to the pier resting upon it, before the bridge was completed. The pier which is here spoken of, being the fourth pier on the Westminster side of the bridge, was undermined below the caisson bottom by the removal of ballast from the bed of the river too near to the foundations. This caused the pier to sink more than 12 inches at one end, and upwards of 13 inches at the other end; in addition to which, the arches on each side of it were so severely fractured that they had to be taken down and rebuilt. These works of reparation caused the opening of the bridge to be delayed for three years. Of late years also considerable expenses have been incurred in affording additional protection to the foundations. It has been said that Labelye ought on no account to have omitted the precaution of driving piles below his caisson bottoms. To those, however, who would impute blame to one who with so few examples to guide him, was obliged to depend so much upon his own unaided judgment, it may be sufficient to suggest, that for any error of this kind the engineer of Westminster Bridge is absolutely undeserving of any censure in comparison with that which is justly incurred by a modern engineer when he commits a similar mistake.

The bridge was completed in 1750, at a cost of £224,857, and opened for public traffic in the same year.

At the present time, something more than ninety years after the completion of the bridge, the increased accommodation required for the passage of vehicles across this great public thoroughfare is about to be provided for by an additional width of 12 feet to be given to the carriage-way of the bridge. For this purpose several coffer-dams have been constructed, within which the southern ends of several of the piers have been lengthened, and it is expected that the operation will shortly be completed along the whole line of the bridge.

[2] For some of these particulars we are indebted to Mr. Wishart's account in the Manuscript Papers of the Institution of Civil Engineers.

FOOT BRIDGE OVER THE WHITADDER, AT ABBEY ST. BATHANS.

Plate 25.

This is a bridge erected from the designs of Messrs. Robert Stevenson and Sons, of Edinburgh, on the property of George Turnbull, Esq., at Abbey St. Bathans, Berwickshire.

This work exhibits an application of the wrought iron tension bar to the purpose of strengthening the horizontal beams which support the road-way. The strength which is produced by the employment of the tension bar is equivalent to the force by which it resists extension in the direction of its length, or it is equal to that power which would be required to cause an extension of the bar. The tension principle has lately been extensively employed in this country, particularly in numerous railway bridges and viaducts. The particular mode in which the bar is disposed, in order that a strain upon the beam shall act as a direct force of tension upon the bar, may be varied considerably from the design now before us; and the more common practice appears to be that of fixing the extremities of the chain somewhat above the upper surface of the beam, and giving it such a curve that it shall not hang below the under surface of the beam.

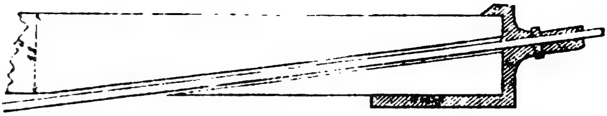
All bridges on the tension bar principle may be considered as a modification of the chain bridge of suspension, since, in common with the latter, they all acquire their strength from the resistance opposed by the iron to the force of extension. The ordinary chain bridge, however, involves the necessity of high piers on which to support the chains in the form of a catenary before the road-way can be suspended below them: considering this, the tension bar bridge presents not nearly so close a resemblance to the common chain bridge as it does to the form of suspension bridge designed by Mr. Stevenson for the river Almond, on the road between Edinburgh and Queensferry. In this latter bridge there was no masonry above the level of the road-way, but the chains were to hang over the top of the abutments, and to be secured at the back of them without high piers of suspension. The road-way was supported *above* the chains by uprights resting upon these, so that this design presents a much closer analogy with the general class of tension bar bridges, in all of which the road-way is above the bar. Again, the tension bar bridge is peculiar in this,—and in this it deserves to be distinguished from all other kinds of bridges resisting by extension,—that the tension bar, instead of being hung as a catenary, independent of the road-way which it supports, is here com-

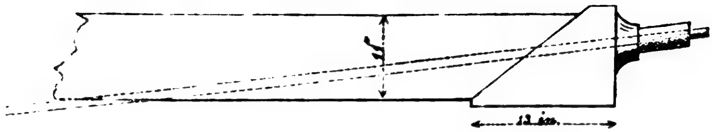
bined with the beam, so as to form the latter into a truss of great strength and rigidity. So completely indeed may the rigidity of the beam be effected by means of the tension bar that, by tightly screwing the latter, the beam may even be forced into a curved or arched form.

DETAIL OF THE BRIDGE.

It consists of two main openings, each of 60 feet span, in which the tension principle is used, and a small land opening of 24 feet, across which the road-way extends from the left-hand pier to the ground on a level with the surface of the latter. On the right-hand side of the river the bridge rests upon rock which serves as an abutment. The width of the road-way is 4 feet, and the entire length of the bridge 160 feet. The piers are built of coursed whinstone: the higher of the two is founded upon rock at a depth of 4 feet below the bed of the river, and the other is founded upon a timber platform laid upon the gravel. The larger pier is 8 feet 6 inches wide at the base, and the other is 6 feet. They both batter to 6 feet in length by 4 in width at the top. A protection of stones is laid round the base of both piers to defend them from the wash of the water. The road-way beams are formed of planks of red pine, 11 inches deep by 3 inches wide. Two of these fixed together form the road-way beams, the dimensions of which are thus 11 inches by 6 inches. The planks are fastened together by oak trenails, placed 3 feet apart, driven entirely through, and wedged at both ends. The beams formed of planks, as described, are of unequal lengths, and are united by scarfs, 2 feet 6 inches in length, which occur exactly over one or other of the uprights which support the beams above the tension rod. At the end of each beam is a cast iron shoe, into which the beam fits. Each shoe is cast with a hole through its back, to admit the tension rod. The tension rods pass through auger holes bored in the beams, and through the holes in the backs of the shoes, and are secured at the back of the shoe by screw nuts, 6 inches long. The screws are used for the purpose of tightening the rods until the beams are quite rigid. The tension rods are 1 inch in diameter, and the annexed figures will show more clearly the mode in which they are passed through the beams, and secured to the iron shoes.

The iron supports for the beams are firmly keyed to the tension rods at their lower extremities, and the tops of the supports receive the beams between two cheeks, through which they are firmly bolted, each with two iron bolts. The supports, four in number in each arch, are opposite to each other, and between each opposite pair are two diagonal bars of ½-inch





iron, which serve to keep the upright supports in their places, and to give lateral stiffness to the bridge. The road-way planking consists of pieces 4 feet 9 inches in length by 2½ inches in depth, and the uprights of the wooden railing are bolted to the longitudinal wooden beams. The whole cost of the bridge was £238.

WEYMOUTH NEW BRIDGE.

Plate 26.

This bridge appears from the Plate to consist of two stone arches, each of 55 feet span, with an iron swivel arch in the centre of 34 feet span.

We regret that in consequence of the death of George Moneypenny, Esq., the engineer of this bridge, we are unable at present to give any details of its construction. Arrangements will be made, however, for supplying the deficiency in a second edition of this work.

HUTCHESON BRIDGE, GLASGOW.

Plates 27, 28, 29, 30, 31, 32, and 33.

This is a stone bridge erected over the River Clyde, from the designs and under the direction of Robert Stevenson, Esq., Civil Engineer, of Edinburgh.

The specification of Hutcheson Bridge being given in full in the first volume of this work, where the reader will also find an historical notice by Lawrence Hill, Esq., Chamberlain of Hutcheson's Hospital, at Glasgow, it will be unnecessary to say any thing in this place by way of description of the bridge.

The specification, it will be observed, is written, as all specifications

necessarily must be, in anticipation of the actual execution of the works, directing the particular forms and manner of construction which *are to be* observed, and affording no information with respect to any departure or variation from the original intentions during the progress of the work. In the present case, however, Mr. Hill's descriptive account supplies a notice of the principal variations made from the specification, or rather it acquaints us with the alternative which was followed under circumstances in which Mr. Stevenson had deemed it expedient to avoid positive directions in his specification. The pile-work of the foundation is here alluded to. On reference to p. 111, vol. i., where this pile-work is described, it will be found that Mr. Stevenson anticipated that the foundation might prove so soft and unsound as to require an increase in the number of piles specified, and particularly to require that the outer row of bearing piles for the masonry to rest upon, instead of being driven as usual several feet apart, should be driven close together, after the manner of sheeting piles. Now, Mr. Hill's account informs us that in the execution of the work this was actually found necessary for the northern abutment, in which the platform was made larger than originally specified; the number of bearing piles was increased, and the outer row of these driven close, as already described. Mr. Hill also states that the sheeting piles in the inner row of the coffer-dam for this abutment were not drawn, but were permanently driven home, as an additional security to the foundation.

Of the seven Plates referring to this bridge, three are of a highly interesting kind, and represent the construction in a way which is somewhat novel, at least in English works. These Plates derive their chief merit from the combination of numerous very clever sketches of figures and machines, with a correct orthographic representation of the building at certain stages of its progress. Thus, in Plate 28, the student is presented with an admirable picture of the condition of the work at the time when the piers and abutments are all carried up to the springing height and the arches are in various degrees of progress. The time here chosen is one of considerable interest and importance in the execution of the work, and none could be better selected for a display of the numerous expedients and resources which are applied in the modern practice of bridge building. The letters of reference in this Plate are accompanied by a brief catalogue of the various parts of the bridge, and of the centering and machines in use; and it will only be necessary, in order to a full explanation, to enlarge a little upon this catalogue.

In the first place, the attention of the student is requested to the service

road-way G G, which is a temporary structure of timber balks laid across from pier to pier, and having two intermediate supports in each arch. The balks composing the service road-way are laid a sufficient width apart to afford support for the wheels of the small waggons *c c c* which travel upon it. A light iron railway is laid down upon the service road-way. Above the service road-way is raised a kind of skeleton scaffolding to support the building-waggons as they are moved in succession over the parts of the arch where the stones are required to be set. This scaffolding rests upon wooden standards raised upon each of the piers and upon intermediate uprights. The scaffolding consists of two lines of logs on that side of the bridge where the service road is laid, and of one line of logs on the other side. When the building-waggons are to be employed over any particular part of the work, two transverse beams are laid down across the top of the scaffolding, and these beams being furnished with iron rails, or round bars of iron, for the wheels to run upon, the building-waggons are easily pushed from one side of the bridge to the other, just over the spot where the stones are to be set in the work. It will be observed that in the operations here represented a sufficient number of men are required to be stationed on the top to work the building-waggons, which it will be observed are merely moveable cranes, and also that there is no contrivance for moving the frames to support these building-waggons. We are therefore inclined to think that although this kind of machinery works very well when once adjusted in its proper position, so that the building-waggons may travel just over their work, yet a good deal of trouble must be occasioned in shifting the frames forward over each successive part of the work. Upon the whole, we greatly prefer the complete machine called a travelling crane, which is furnished with small wheels for travelling on the service road, and in which there are two separate powers, one for causing it to advance backwards or forwards, and another for moving the building-waggon into any required position. In the most complete travelling cranes also the men are not required to be mounted on the top of the scaffold, but work on a stage at the same level as the service road itself.

The scaffolding in Plate 28 is only shown completed over three of the arches, but the dotted lines over the other two indicate its continuation entirely across the bridge. In the first arch on the left-hand side the men are employed in fixing the long beams of the centering by means of sheer poles, ropes, and pulleys. In the second arch the ribs of the centering are all fixed, and men are carrying battens on both sides of it to form the

covering for the reception of the arch-stones. In the third arch a number of the courses are set on each side, and a weight D is placed on the centre to balance the pressure laid upon the haunches of the centering. In the fourth arch the building-waggons are seen in operation, setting the stones in their places, and the fifth arch is shown complete.

At each end of the bridge, stones for the arches are arriving in trucks, and are being removed by cranes into the railway waggons, which are pushed along the service railway till they come under the building-waggons, ready to be hauled up and lowered down to their proper places in the work. The implement by means of which the stones are raised and held in suspension to be lowered on to the railway waggons, is well known under the name of the *Devil's claws.* Its construction is such that the greater the weight of the stone the greater is the force with which the claws are pressed into it, and hence its value for purposes of this kind where the masses to be moved are very heavy, and where the consequences of a fall would be very serious.

Plate 29 shows the building apparatus as seen in a cross section through one of the arches. The width of the service road-way is here shown, also the frame for supporting the building-waggon, and one of the standards supporting the scaffolding.

The remaining Plates of this bridge will be so clearly understood by the aid of the specification as to require no further explanation.

BRIDGE OVER THE RIVER SCHUYLKILL, AT MARKET STREET, PHILADELPHIA.

Plates 34 and 35.[3]

This is a wooden bridge of 494 feet water-way, erected upon stone piers across the River Schuylkill. The bridge was commenced in 1799, but was not completed till 1805. It consists of one centre arch of 194 feet span, and two side arches of 150 feet each.

The River Schuylkill at this place is of considerable depth, being as much as 28 feet at low water. The bed of the river is composed of soft alluvial mud, in consequence of which it was necessary to excavate down to the solid gneiss rock which extends under the river. The foundation

[3] The reader will perceive that the scales to both these Plates are figured wrong. In Plate 34 the figures 20 and 40 should be respectively 100 and 200 feet; and in Plate 35 the figures 1, 2, 3, and 4, should be replaced by 5, 10, 15, and 20.

of the western pier is 41 feet 9 inches below high-water mark, and that of the other pier is 26 feet below the same level.

In founding the deepest of these piers a coffer-dam of very large dimensions was required. The western pier is 35 feet in thickness at the foundation, and 25 feet at the level of high water, the masonry being carried up with a batter of an inch and a half to the foot. At the level of high water is a set-off which diminishes the thickness of the pier to 22 feet; this thickness decreases with the same batter as below, till at the under side of the springing course it measures 19 feet. The other pier is 33 feet in thickness at the foundation, and 25½ feet at the level of high water, the batter being the same as in the other pier. The thickness under the springing is also the same in both piers. The ends of the piers are semicircular, and the whole height and thickness are built of solid masonry. The length of the piers from end to end at the level of the springing is 66 feet.

The abutments and wing walls are built perpendicular, without buttresses. The abutments are of solid masonry 18 feet in thickness; the wing walls are 9 feet thick at the foundations, their thickness diminishing by offsets to 18 inches at the parapet. The eastern abutment and wing walls (right-hand side of the Plate) are founded on rock, but those on the western side are built on piles. The masonry of the piers is strengthened by numerous chains laid across them in various positions.

The timber superstructure consists of three frames in each arch. These frames have been designed on the principle of imitating by the timber-work the arrangement of the voussoirs in a stone arch. Thus the whole space between the curved soffit of the arches and the upper line of the framing is divided by the radiating queen-posts into a series of compartments exactly corresponding with the arch-stones in a stone bridge; and the two halves of each adjacent pair of queen-posts, with the diagonal brace between them, may be considered as a separate voussoir.

The details of the framing will be understood on reference to Plate 35, in which the corresponding parts are denoted in each figure by the same letters of the alphabet. It appears that the lower or curved part of the frame, namely, that which is called the rib in Plate 35, is composed of two pieces, each 16 inches in depth by 14 inches in breadth; and these pieces are framed close together and bolted at short intervals with two bolts, as shown in Plate 35. At distances of 17 feet on this rib a groove is cut in each of the timbers composing it, to receive the end of the queen-post, as shown by the perspective sketch of D in Plate 35. The queen-

post has corresponding grooves, as shown by the sketch of E, and the timbers of the rib are secured to it by two trenails, as shown in the side view. Pieces of timber are framed in between the bases of the queen-posts, so as to rest immediately upon the top of the ribs, and thus aid in resisting the thrust of the diagonal queen-braces upon the shoulders at the base of the queen-posts. These timbers are shown in the side view and in the general elevation. The chords, which might, be also termed the longitudinal timbers for supporting the road-way, are composed, like the ribs, of two separate logs, bolted together at short intervals. These logs are each 14 inches square, and wherever they are crossed by the queen-posts, part of their thickness is cut out of each in the same way as out of the ribs, and as shown by the sketch of G in Plate 35. Like the ribs, they are further secured to the queen-posts by two trenails at each crossing. The ends of the road-way girders are seen resting upon the chord in the general elevation and in the side view, and they are shown lengthways in the cross section. Along the top of the queen-posts is laid a log of timber 14 inches square, marked *a* in Plate 35. This log, called the plate, is firmly secured by an iron strap to the head of each queen-post. Immediately beneath this plate, pieces of timber are framed between the tops of each of the queen-posts, to serve the same purpose of resisting the thrust of the queen-braces, as already described with respect to the corresponding timbers at the foot of the queen-posts. Resting upon the plate, pieces of timber (B) are laid across the bridge from frame to frame at the tops of each of the queen-posts. These pieces serve as the straining or tie-beams for a roof where necessary. Part of the framing of a roof is shown in the cross section, but no part to correspond appears in the other views. The brackets between the cross pieces (B), the bolts by which these brackets are attached, and the iron straps securing the plates to the heads of the queen-posts, are all shown in the side view and cross section of Plate 35.

It will be seen from the plan, Plate 34, that the bridge has three frames; and as the centre frame, like those of the outside, rises considerably above the level of the road-way, being, in fact, precisely similar to the outside frames which have been just described, it is evident that the road-way is thus divided longitudinally into two parts, from one to the other of which carriages are unable to cross. To compensate for this, the width of each division is sufficient for the passage of two carriages abreast. The space across the bridge between the centre and each of the side frames is 18 feet 6 inches, of which 5 feet on each side of the

bridge consists of a foot-path raised above the carriage-way, and protected by posts and chains. Thus the width of each carriage-way is 13½ feet.

The following additional particulars of measurement may be useful to those who wish to avoid repeated reference to the Plates.

PARTICULARS.

	feet.
Length of the bridge superstructure	550
Length of abutments and wing walls	750
Span of small arches, each	150
Span of centre arch	194
Width of the bridge	42
Versed sine or rise of centre arch	12
Versed sine or rise of small arches	10
Versed sine or rise of chords	8
Height from the surface of the river to the carriage-way	31
Height in the clear over the carriage-way	13
Thickness of each pier	20
Length of each pier	62
Depth of water on foundation of western pier	41¾
Depth of water on foundation of eastern pier	21

The whole cost of this bridge was $295,000, which includes a sum of $45,000 paid to the corporation of the city of Philadelphia for the site of the structure.

The superstructure was designed and erected by Timothy Palmer, of Massachussetts; and the coffer-dam, which was a very difficult and extensive work, was designed by William Weston, an English engineer.

CHEPSTOW BRIDGE.

Plates 36, 37, 38, 44, 45, 46, and 47.

This is a bridge erected over the River Wye by Messrs. Rastrick and Hazledine, of Bridgenorth, at the joint expense of the counties of Monmouth and Gloucester, which at this place are separated by the river. The bridge consists of five arches, namely, a centre one of 112 feet span, two side arches of 70 feet span, and two others of 34 feet each. The piers and abutments are of dressed ashlar masonry, the stone having been principally procured from the quarries at Whitchurch, in the forest of Dean. The arches are formed by cast iron ribs, and surmounted by a handsome wrought iron railing.

Amongst the rivers of this country, few are more celebrated for the extreme beauty of their scenery than the Wye in the neighbourhood of Chepstow. Through mighty clefts hewn in the old red sandstone and mountain limestone of the district by gigantic natural forces, this river pursues its course from Monmouth to the Severn, into which river it falls about a mile below Chepstow. From one of the magnificent cliffs which overhang the vale of the Wye, its winding course may be seen for many miles, and traced with almost as much minuteness of detail as on a map. At one place, bounded by high and upright naked rocks, the river below shoots rapidly past, and at another, sweeping into a wide and magnificent amphitheatre of nature's own creation, the solid walls of limestone which encircle it are clothed with the most luxuriant growth of ivy; while, far above, the forests of noble timber give to the scene the last and only heightening beauty which it wanted to rival the finest creations of fancy.

The venerable monastic pile of Tintern Abbey—the gorgeous view from the summit of the Windcliff—and the mouldering ruins of Chepstow Castle, are objects which he who once has seen them will long remember with interest. In addition to its natural beauties, however, the Wye is distinguished by some of the most remarkable tidal phenomena in the world. The broad wave which rolls up the Bristol Channel with a perpendicular head of 4 or 5 feet exhibits a fine example of the phenomenon called the bore. This is confined to very few rivers in the world, and it always indicates a very strong and powerful energy in the tidal wave. Now the River Wye, where it joins the Severn, has a very contracted mouth, and the tide flowing up is almost immediately confined between precipitous cliffs of limestone, which act as a partial dam, and produce the usual effect of a dam, namely, that of throwing up a great head of water. The rise of tide, therefore, which in the Severn, near the mouth of the Wye, is about 26 feet, is suddenly increased in the Wye, till at Chepstow Bridge an extraordinary spring tide rises about 45 feet. It is commonly stated in guide books, and related to visitors as a piece of information, that the tide here rises 55 feet, and that on one occasion, not many years ago, it rose to the height of 70 feet. A reference to the elevation in Plate 44 will at once show that the former of these statements is erroneous, because if we set off a height of 55 feet above low water it will reach above the crown of the centre arch, and will inundate to a depth of many feet the whole road-way on each side of the two small piers, and far beyond the extremity of the bridge. It will scarcely be necessary to say that nothing of this kind happens at ordinary spring tides; at the same time it

will be evident that this must be the case whenever the tide rises 55 feet. The other tale about an extraordinary rise of 70 feet is still more incorrect and improbable, for the water in this case would have been 14 feet above the highest part of the road-way at the crown of the centre arch, and would have inundated several hundred houses in the lower part of the town, to the height of their second story. The writer of this account has ascertained from personal inquiry that no such inundation has taken place since the present bridge was built, nor as far before that time as the memory of man can reach. He is therefore inclined to view the whole story as a fabulous account of some tide which rose about 5 or 6 feet higher than the line of high water shown in the Plate; this additional rise, it should be observed, being one which would do considerable injury to the town, and be likely enough to give origin to exaggerated accounts. The line of high water shown in the Plate should have been noted as that of an extraordinary spring tide, as it seldom rises higher than 3 feet above the iron caps of the piers; whereas in the Plate the line is drawn 6 feet above these caps. We are therefore inclined to think that the tide rises very little more at Chepstow than it does in the Avon at Bristol, where the highest is said to be about 40 feet. Both rivers are in similar circumstances with respect to the sudden lateral contraction of the tidal wave, an effect which, as its area remains the same, necessarily forces it to assume a greater vertical dimension.

FOUNDATIONS.

The abutments on both sides of the river are founded on the limestone rock.

The foundation of the small pier on the Monmouthshire side consists of a very wide starling, formed in the bottom of rubble stones thrown in upon the remains of a former pier, and finished off at the top by a surface of squared stones intersected by a frame-work of timber, of which the form and dimensions are shown in Plate 36. All the timbers are bolted at the joints and crossings by iron pins. A row of sheet piling was originally driven down in front of this starling, but it appears from a remark on Plate 36, dated June, 1839, that this sheet piling was at that time entirely destroyed. On seeking for an explanation of this fact, we have been favoured with copies of Reports addressed to the magistrates of both counties by Mr. Samuel Hughes, Civil Engineer, in which the original cause of the destruction is referred to the great velocity of the current in the very contracted water-way between this starling and that of the next adjoining pier.

It will be seen from inspection of Plate 44, that the water-way is here contracted to less than half the span of the arch, and for at least two hours of every tide the water is confined to this contracted channel. The Report goes on to show that a breach had at length been made in the upper angle of the starling, and when once the water found its way into the interior of the starling, the injury which it occasioned appears to have been very rapid. By successive degrees, small pieces of mortar and stone were carried out, and a few of the piles became loosened. The destruction proceeded, till at length every pile, with the exception of about half a dozen at the lower or down-stream end of the starling, was carried off by the force of the current. At the same time the water which found lodgement inside the starling appears to have exerted some kind of hydrostatic pressure upon the squared stones and the timber composing its surface, as these were much displaced and tilted up from their beds at the time of Mr. Hughes's examination.

The Report concluded by recommending that the starling should be so reduced in size as to have no greater projection into the water-way than those of the other piers, and the necessity of executing this work without delay was forcibly pointed out. We are informed that although the presiding magistrates at the Quarter Sessions of both counties were perfectly convinced of the accuracy of Mr. Hughes's views, no steps have yet been taken to remove the very injurious obstruction which this starling presents to the free course of the river.

It was understood at the time of Mr. Hughes's Report in 1839, that the heavy rates entailed on the county of Monmouth, by the riots at Newport, formed a serious objection to an advance of funds for any alterations of Chepstow Bridge; and the county of Gloucester appears to have considered that its rates could not properly be used for alterations of the pier on the Monmouthshire side, this being situate considerably more than half-way across the river, and therefore within the county of Monmouth. Although it is probable that the embarrassment of the rates applicable to works of this kind has been retrieved by this time in Monmouthshire, the objectionable starling still remains,—a circumstance much to be regretted, as it undoubtedly exposes the bridge to great injury, and might even occasion the entire destruction of the pier, if the river, during a hard winter, were much obstructed by ice.

The starling of the corresponding small pier on the Gloucestershire side contains no timber, and is built of roughly-squared stones, set without mortar, to the height of five feet above low-water mark, at which height the regular masonry of the pier commences.

The starlings of the two centre piers were built within encaissements of piling, and as the great height to which the tide rose rendered the construction of a coffer-dam a work of too great expense, the difficulties experienced in getting in the foundations must have been very great. Plate 45 shows the wood-work of the starling for one of these piers; and it may be remarked, that although the arrangement of the timbers is somewhat different in these two piers, yet they are both in equally good order, and exhibit no traces of that injury from which the small pier has so considerably suffered.

The dimensions of the wood-work are so clearly shown on the Plate that it is only necessary to add that the pieces were principally of the same depth as their breadth. The dovetail joints are mostly secured by iron pins, and where the diagonal and cross timbers are let into the centre logs, they are secured by oak trenails, 1 inch square.

PIERS AND ABUTMENTS.

These are built of regular squared masonry, the stone used being a hard-grit sandstone, procured, as before mentioned, from the forest of Dean. The piers are carried up with a batter, as shown in the elevation, Plate 44. Each of the piers has an iron cap, screwed down to it in a substantial manner. This cap overlaps the whole outline of the pier to the depth of 18 inches. It is formed of metal, 2 inches in thickness, and cast with hollows at the pier point, and in the spaces between the ribs of the arches.

IRON-WORK.

Each of the piers supports five iron standards, of a triangular figure, with flanges on the outside, to which the ribs of the arches are bolted. One of these standards is shown in detail in Plate 47, with the nuts, washers, and screws used for bolting the ribs to the standards. The Plate shows a standard for the interior ribs, with plain flanges: those for the outside ribs have a flange on the inside only, a rounded moulding on the standard supplying its place on the outside. The elevation and plan in Plate 46 show this difference in the mode of fixing the inside and outside ribs. The round cylindrical bosses on the plan of the standards indicate the points at which these are fastened to the pier and its cap by powerful iron cramps. These bosses are also shown in the elevations of the standard. The plan in this Plate shows also the number and position of the nuts and screws for securing the iron cap to the pier. In the two large piers these screws are 2 inches in diameter, the nuts 4 inches square, and the washers 6 inches diameter. In the small piers the screws are 1¾ inch diameter, the nuts 3¼ inches square, and the washers 5 inches diameter. The ribs are quite plain, and without any

flanges, except those already mentioned for bolting them against the standards. Plate 44 shows the general shape of all the ribs, with the proportion of hollow and solid parts. The elevation in Plate 46 shows on a larger scale part of two ribs springing from a small pier. Plate 47 shows the method of bolting them on a still larger scale. Plate 37 is a transverse section through one of the large piers, showing the whole five ribs, and Plate 38 is a plan and elevation of one of the large piers.

The screws for securing the ribs to the iron standards are 1¾ inch diameter. The heads of these screws are 4 inches square and 2 inches deep; the washers are 4 inches diameter and 1 inch in thickness, and the octagon nuts are 3½ inches diameter and 2 inches deep.

Each rib was cast in two parts, and bolted through the key pieces which run across the bridge under the road-way plates at the crown of each arch.

To secure the ribs in a vertical position, and to give lateral stiffness to the bridge, there are horizontal through bolts of wrought iron passing through all the ribs at every alternate angle of the hollows in the lower part of the rib. There are twenty of these through bolts in the centre arch, twelve in each of the side arches, and seven in each of the end arches. These bolts are 1 inch in diameter, and pass through wrought iron tubes, 3¼ inches outside diameter. These tubes have collars abutting against the sides of each rib, to keep them firmly in their places. The bolts have heads 3½ inches square and 1⅞ inch deep, and are terminated at the other extremity by octagonal nuts. A series of diagonal braces are screwed by flanges between the adjacent pairs of ribs from one collar of the tubes to the springing of each rib at their opposite corners, and also between the collars at opposite corners, as shown in the plan, Plate 46; and in Plate 47 will be found a cross section of one of these diagonal braces. The road-way plates resting on the top of the beams are ¾ of an inch in thickness and 3 feet in breadth. The iron cornice is bolted to the road-way plates, and the standards of the railing secured to the cornice, as shown in the section, Plate 37. The whole of the iron-work is put together in a most substantial manner, the joints are all made sound and water-tight by a powerful metallic cement, and every year the whole of the iron-work is painted over, to preserve it from rust.

ROAD-WAY.

The width between the railing is 21 feet, of which 6 feet are occupied by a foot-path on each side, leaving only 15 feet for carriage-way. This is much too narrow for the traffic, which would have been much better

accommodated if the bridge had been constructed with six instead of five ribs.

LONDON BRIDGE.

Plate 39.

(See Hosking's Practical Treatise, vol. II. p. 229, &c.)

BRIDGE UNDER ROAD FROM CROYDON TO SYDENHAM, ON THE LONDON AND CROYDON RAILWAY.

Plate 40.

This is an oblique bridge, built on an entirely novel principle from the designs and under the superintendence of Joseph Gibbs, Esq., engineer of the London and Croydon Railway. The whole bridge is of brick-work, and instead of the arch being built in spiral courses, as in the common skew bridge, it is composed of four ribs or square arches, 3 feet in length, and the direction of the courses in all these ribs is at right angles to the line of the road-way over the bridge. It is evident that a bridge may be built on this principle at any angle, however acute, and all the trouble and expense of the skew arch avoided. The bridges of this kind on the Croydon Railway are universally admired for their elegance and simplicity.

The present bridge is built across a cutting about 18 feet in depth. The wing walls step up the slopes, and are continued on one side of the bridge to a considerable length, namely, as far as the point shown on the plan where the approach diverges into two separate roads, and at which point the raised approach meets the natural surface of the ground.

The abutments are founded 3 feet below the level of the rails and are carried up solid for a height of 5 feet, at which height each abutment consists of a rhombus whose extreme length is 28 feet 6 inches with a breadth at right angles to the face of 9 feet 6 inches. This lower part of the abutment forms a platform on which four low walls of brick-work are raised as the real abutments of the four ribs of the arch. The walls are rectangular, being 32 feet 6 inches in length by 3 feet in breadth, and the same in height. The two outside walls are continued to form the wings, as shown in the plan.

The arches are elliptical, 53 feet in span, with a rise of 12 feet. They are four bricks in thickness, and their length, measured at right angles to the road-way, is 3 feet, the same as that of the small abutments from

which they spring. The outsides of the brick arches are covered with slates, as shown in the transverse section, and upon these is placed the metalling of the road-way. The wing walls are divided by pilasters into several panelled compartments which accord well with the character of the arched ribs. The parapets are 4½ feet in height above the road-way, which is 20 feet in width.

BRIDGE UNDER ROAD FROM NORWOOD TO BROMLEY, ON THE LONDON AND CROYDON RAILWAY.

Plate 41.

This is an oblique bridge by the same engineer as the last described, built also on a novel principle, although entirely different from that in Plate 40. The main arch here springs from the slope of the cutting, and is built on the skew principle; but the peculiarity consists in the pier of brick-work standing in the usual place of an abutment in bridges of this kind, and appearing in the elevation to cut the arch in two at a perpendicular raised from the edge of the ballasting. The arch thus appears at first sight to be of a much smaller segment of a circle than it really is, to have a less versed sine, and to be flatter; and this delusion ceases not till the continuation of the arch is observed on the other side of what seemed at first to be the abutment. In this bridge there are three small arches on each side which spring from small piers 2 feet 3 inches in thickness, and founded on the slopes. The main arch is three bricks thick, 31 feet 6 inches span, measured on the skew line, and 30 feet measured square across, that is, at right angles to the line of railway. The small piers are built solid without openings, but those which intersect the main arch have each two arched openings 9½ feet in extreme height, and 5 feet in width, as shown in the elevation, plan, and transverse section. The half longitudinal section in this Plate is a section at right angles to the railway, and shows the solid filling in between the small arches, also the paving on the slopes between the bases of the small piers. The wing walls extend a very short distance between the last of the small arches, and are curved outwards at the ends, as shown on the plan. The parapet is four feet in height, with a neat plinth and coping.

The road-way over this bridge is 30 feet in width between the parapets, in consequence of which a length of 33 feet is required for all the arches.

BRIDGE AT SYDENHAM, LONDON AND CROYDON RAILWAY.

Plate 42.

This bridge, built partly of brick and partly of stone, has a centre arch and side arch formed of flat cast iron girders to carry a road over the railway in 13 feet cutting. The centre arch is 30 feet span, and the side arch 11 feet. The pier is of plain brick-work with stone quoins, and is founded upon two courses of footings. It is 5 feet in thickness and 15 feet in height from the surface of the rails to the under side of the girders. The abutments and pier are each 29 feet in length. The abutments are 4 feet in thickness, with two counterforts of 3 feet square in each. The wing walls on one side of the bridge are very short. On the other side the wings are 19 feet in length, with a return of 6 feet at the end, as shown in the half plan of foundation. The darker shading in the half plan and half longitudinal section indicates those parts of the bridge which are built of stone. The entablature is also of stone, and the whole of the work, both brick-work and masonry, is set in Roman cement.

The seven interior girders are plain ribs of iron, 18 inches in depth in the centre and tapering off to each end, as shown in the half longitudinal section. These girders have each a flange 12 inches wide at the bottom, but no flange at the top. The outside girders are double, with a flange on the inside, but plain on the outside.

The road-way plates rest on the flanges of the girders, and the parapet walls rest on the tops of the outside double girders. The parapet is 14 inches thick and 4 feet in height above the road-way. This is a cheap and substantial bridge, and has been much admired for its neat and chaste appearance.

VICTORIA BRIDGE, ON THE DURHAM JUNCTION RAILWAY.

Plate 43.

This bridge is the principal work on the short line of the Durham Junction Railway, which was intended to connect the Stanhope and Tyne Railway with the north end of the Hartlepool Railway. At present, however, the Durham Junction Line is only completed as far as the Seaham Railway, in the township of West Rainton, its whole length from the Stanhope and Tyne Railway to this place being $4\frac{7}{8}$ miles.

The Victoria Bridge or Viaduct, which forms the subject of Plate 43, carries the railway across the valley of the river Wear, near Low Lambton. This bridge consists of four large arches, two of which are 100 feet in span, another is 144 feet, and the largest, being the arch across the river in the deepest part of the valley, is 160 feet. Besides these four large arches there are three small arches of 20 feet span at each end of the bridge. The whole length is 811 feet, and the width across the bridge from out to out is 23 feet 4 inches.

The three piers and one abutment are founded on solid rock, and required no artificial foundation. They have each three courses of footings, which taken together measure 5 feet in depth. Above the footings the piers are carried up to the springing with hollows, in order to diminish the expense of solid masonry. The southern abutment is founded on piles. These are of Scotch fir, about 14 feet long and 10 inches diameter, driven about 3 feet apart. The sleepers, which are bolted down to the pile heads, are covered with two thicknesses of 3-inch Memel planking. The middle pier is 23 feet 9 inches in width, and the two other piers each 21 feet 6 inches below the springing of the two middle arches. The piers are 54 feet in length from point to point of the cutwaters, and above this height the width across the pilasters from point to point measures 31 feet.

The two middle arches spring from the same level. The smaller of these two arches is a semicircle of 144 feet span, and the larger a segment of a circle 160 feet in span with a rise of 72 feet. The two small arches of 100 feet span are both semicircular, and spring from a line 22 feet 6 inches above the springing of the large arches. The quoins of these four large arches are of Aberdeen granite. The dimensions of the small side arches and their piers will be seen from the Plate, which contains the actual measurements taken from the engineer's working elevation. The string course is a perfectly horizontal line from end to end, and its height is 120 feet above high water of spring tides in the river. The spandrils are quite plain on the face, and the space between them consists of interior spandrils and cross walls, which are flagged over to receive the ballasting of the railway.

The whole of the bridge, with the exception of the granite quoins of the large arches, is built of stone procured from Pensher quarry, about a mile and a half from the bridge.

The Pensher stone is a pale whitish-brown sandstone, composed of coarse quartz grains, united by an argillo-calcareous cement, and interspersed with plates of mica. It has been employed in many important

buildings, as in the Scotch Church at Sunderland, St. John's Chapel, Bishop Wearmouth, Wynyard Mansion-house, and in the works at Sunderland Pier and Seaham Harbour.

The amount of the contract sum for building the Victoria Bridge was £34,619, and its actual cost £38,000.

BRIDGE OVER THE TYNE, AT SCOTSWOOD.

Plates 48, 49, and 50.

This is an oblique bridge built over the River Tyne, at Scotswood, in the line of the Newcastle and Carlisle Railway, from the designs of John Blackmore, Esq., the present engineer of the company. It consists of eleven openings, each of 60 feet span, measured on the skew line; and the height at which the railway is carried across the river is 35 feet above low water. The railway at this place forms an angle of 55° with the course of the river, and the piers being built coincident with the course of the river, the bridge stands at an angle of 35° with the direction of a rectangular crossing. With the exception of the abutments, the whole of this bridge, from the foundation piles to the top of the railing, is built of timber. One of the abutments is founded upon piles, the other upon a bed of concrete about 10 feet in thickness. Each pier consists of twelve piles, namely, five in length and two in width forming the body of the pier, with a cutwater pile at each end. The small black squares in the plan, Plate 48, show the position of the piles in the piers. The twelve long piles in each pier are 12 inches square, 33 feet in length, and are driven into the ground to an uniform depth of 22 feet below low water. At a height of 6 feet above low-water mark the double row of piles in each pier is surrounded by a waling of 1 foot by 6 inches, fixed with its least dimension vertically, and bolted through each opposite pair of piles, as shown in the transverse section in Plate 50; and this waling extends to the cutwaters at the ends. At the top of the piles, that is, 6 feet above high-water mark, is fixed a similar waling, which is bolted down to the head of each pile.

At the level of the lower waling is inserted between each opposite pair of piles a piece of timber, the ends of which are notched into each pile, as shown in the transverse section, Plate 50. A wrought iron through bolt passes through the waling on each side, and through this piece of timber longitudinally. This piece of cross timber serves as the base for an upper series of piles to rest upon. These upper piles are

20½ feet in length and 12 inches square, except for the first 9 feet of their length, in which half their thickness is cut away where they overlap the lower piles. The shoulder formed by the whole timber above the part thus cut away rests upon the head of the lower pile, as shown in the transverse section. At the level of the top of the lower piles is inserted another piece of cross timber between each pair of upper piles, and longitudinally through this piece and through the upper walings before mentioned, passes a through bolt similar to those at the base of the upper piles. Longitudinal sleepers 1 foot square rest on the heads of the upper piles, and between these sleepers at every point where they rest upon the piles is a piece of cross timber and a through bolt, similar to those already described. The longitudinal sleepers are covered by 3-inch planking laid close together across the pier, 5 feet 8 inches in width and 38 feet in length. In all the piers the whole of the ten piles which compose the rectangular figure of the pier have upper piles spliced to them, as above described, in order to carry the platform of sleepers and planking for the arches to spring from. The cutwater piles, however, are no higher than the under side of the upper waling, and have no upper piles spliced to them. Between the two rows of upper piles in each pier, diagonal braces extend from the under side of the longitudinal sleepers at one end of the pier to the upper side of the lower waling at the other extremity of the pier. These diagonal braces are shown in the side elevation of pier in Plate 49.

The superstructure resting on the piers is composed of seven trussed ribs over each opening. The elevation and plan of one of the inner trusses is shown in Plate 50. Each truss abuts against a solid wooden buttress composed of three logs of timber 9 inches wide, which rest upon the planked platform of the pier. This buttress is constructed of a slightly tapering form, as shown in the elevation, Plate 50. The lower log is 12 inches in depth, the middle one 6 inches, and the upper 9 inches. The top of the buttress is 4 feet 6 inches longitudinally, and its base 5 feet 6 inches. On the tops of these buttresses, along the whole length of each pier, are laid two whole logs 13 inches square, on which rests the single horizontal beam of each truss. This horizontal beam is 12 inches in depth by 9 inches in breadth, and is rendered inflexible by three struts which are let into it at different angles. These struts abut against the three logs of the small wooden buttresses, and where they are let into the beams they are bolted by wrought iron bolts, which pass through an iron cheek on each side of the beam.

The outside truss beams are of more complicated construction, and the timbers composing them are arranged according to a system which converts the whole parapet railing into a stiff and rigid framing. This arrangement, which adds greatly to the strength of the bridge, is on the same principle as that which Mr. Blackmore has followed in the Ladykirk and Norham Bridge, described at p. lxxxi. The standard or buttress for these outside trusses to abut against is the same as already described for the inner trusses, but there is this difference in the two, that the horizontal beam in the outside truss does not rest immediately on the same longitudinal logs as the inner trusses, but upon blocks of 12 inches in depth, which are interposed to give an additional height to the outside beam. The undermost strut for the outside truss abuts against an under log of timber, instead of being let into the long beam itself, and the other two struts are let in, as shown in Plate 50, for the inner trusses. Above the long horizontal beams of the outside trusses are diagonal struts, which abut against the angles made by the vertical suspension posts with the horizontal timbers of the railing. The enlarged section in Plate 50 exhibits very clearly the transverse dimensions and positions of these horizontal timbers and struts of the railing. The suspending pieces occur at the intervals shown in the side view, Plate 49, and are secured by iron straps to the beams and struts of the trusses and to the timbers of the railing.

The road-way is composed of transverse logs resting on the tops of the inside trusses. To these transverse logs the railway sleepers are bolted, and the spaces between them are covered by 3-inch planking.

BRIDGE OVER THE RIVER EDEN, AT CARLISLE.

Plates 51, 52, and 52ᵃ.

This is a bridge of five semi-elliptical arches, erected from the designs of Sir Robert Smirke. The arches are each 65 feet in span, with a rise of 21 feet, and the whole length of the bridge from end to end is 436 feet.

The piers have each three courses of footings. These are of ashlar stone, each course being 18 inches deep, and projecting 8 inches all round beyond the next ascending course. The lowest of these courses is founded 6 feet below the level of water in the river, as shown in the elevation, Plate 51. The plinth of each pier is built upright in two 18-inch courses of hewn stone. This plinth is 10 feet 6 inches wide, and is chamfered

off at the top to a width of 9 feet. The length of the rectangular part of the plinth in each pier is 36 feet, and each end is terminated by a semicircular cutwater. Above the plinth the cutwaters of the piers are carried up with a slight batter to a height of 6 feet 6 inches above the plinth, at which height they are 8 feet 3 inches in breadth. Each cutwater is surmounted by a stone cap, dressed to a point at the top, and having its under edge chamfered off, as seen in the elevations.

The abutments are built with footings similar in number and depth to those of the piers. Their length is also the same, and their face corners are worked off to the same radius as the cutwaters of the piers. At the level of the plinth courses the extreme breadth of the abutments is 38 feet; but the mass of the abutment is much reduced by the introduction of a counter arch, shown in the plan, Plate 51. This arch, which has a chord of 33 feet and a versed sine of 10 feet, reduces the thickness of the abutments to 20 feet at the centre, and leaves the ends of the abutments as mere walls 8 feet thick, which may either be called wings or considered as part of the abutments. The external face of each abutment has two projections or pilasters, which are continued up to the string course, and correspond with similar pilasters in the parapet wall. The corners of the abutments are carried up in a rounded form to correspond with the extremities of the piers, and at the level of the caps of the piers each of these corners is surmounted by a half cap, similar to those of the piers.

The arches spring from their corresponding piers and abutments at the level of the top of the plinth already described. In each arch the under side or inner curve is described from five centres, the position of which is indicated by the lines B B B B on Plate 52. The outer curve of the arches is described from three centres, two of which are situate on the lines G G at the level of the springing, and the other, from which is described the middle part of the curve, is of course the intersection of the lines G G in the versed sine produced. The arch-stones average 18 inches in thickness, and are 7 feet 4 inches in depth at the pier caps, and 3 feet 9 inches at the crown. All the stones composing the external face of the arches are worked on the under side to a templet to suit the curve of the arch, and their tops are wrought square, so as to form rectangular joints with the courses of stone in the spandrils. The form and general effect of the string course, the parapet, and its pilasters, will be seen from the elevations in Plates 51 and 52; but owing to the absence of transverse sections we are unable to give the thicknesses

of these parts of the bridge, and the same remark applies to the spandrils and to the end walls of the abutments. The string course has a projecting torus moulding, the simplicity of which harmonizes well with the general character of the bridge. The parapet is 3 feet 6 inches in height to the under side of the coping, and is built in three courses, two of which are 15 inches, and the upper one 12 inches, in depth. The coping is 16 inches in depth, with the upper 4 inches bevelled off, as shown in the enlarged elevation, Plate 52. The caps of the pilasters are 3 feet 1 inch in extreme depth, their lower part corresponding with the coping of the parapet, and the top dressed to a point, as shown in Plate 52.

Plate 52[a] contains a plan and elevation of one of the ribs used for the centering. The ends of the under connecting beams are here shown resting on the footings of the piers. These beams are 9 inches square, and extend the whole length of the piers and abutments. The supports of the rib are three bearers or struts on each side, resting one upon each of the connecting beams. These struts radiate to a point within the pier, and the middle one, being required to support the greatest weight, is made of greater scantling than the others. This middle strut is 12 inches square and 4 feet 4 inches long; the lower one is 11 inches square and 6 feet long, and the upper one is 9 inches square and 3 feet 3 inches long. The upper connecting beams rest on the tops of these struts, and their ends, like those of the lower ones, are shown in the Plate. Upon the tops of these upper connecting beams rest the wedges. These have an aggregate thickness of 3 feet 9 inches, and are cut with notches in such a way that when the middle wedge is driven outwards the upper wedge is lowered and the whole centre slackened. On the upper wedge is fixed a cast iron shoe, with four compartments for the reception of the radiating beams which compose the rib. This plate is 9 feet 3 inches in length by 18 inches wide, and $\frac{3}{4}$ inch thick; a plan and section of it are shown at the foot of the Plate. The long horizontal beam A is called the crown tie-beam of the rib; it abuts at each end against a crown piece which rests upon the wedges, and forms with these crown pieces a separate truss, as will be seen by examining the Plate. This beam is 12 inches square, and its plan is shown in Plate 52[a]. Beneath this beam is an under truss, the crown of which rests upon the small beams K K. The crown of this under truss is 14 feet 3 inches in length, and is secured to the tie-beam A by stirrup irons and wedges. The rafters of this under truss are 12 inches square, and rest in the lower

compartment of the shoes already described. The principal rafters, one of which is marked B in the Plate, abut against a crown piece which occupies the middle part of the rib, forming another separate truss. The crown of this truss is 7 feet 9 inches long, and is secured to the principal rafters by bevelled joints. This truss is further strengthened by two king-posts, one on each side. These king-posts are secured to the crown of this truss, to the tie-beam A, and to the crown of the lower truss, by straps and through bolts, three in each strap, as shown in the elevation and plan. The rafters B in this truss being double, that is, one on each side of the crown tie-beam, are of small scantling, namely, 12 inches by 6 inches. The beam marked C in the elevation is one principal of another truss. Its other principal is notched into the crown tie A, and made fast with strap irons and bolts. The beam marked C belongs to the truss on the left-hand side of the arch. On the right-hand side is another truss, which, being similar, the same description applies to both. The king-posts G G, as well as the principals B and C, are double, namely, one on each side of the beam A. These king-posts are of timber 12 inches by 9 inches, and are firmly secured to the crown pieces and to the several beams which they cross by bolts and straps, as shown in the plan and elevation. The pieces K K are intended to connect the ribs with each other and to fix the ribs in their proper position. There are couples of these pieces at the foot of each king-post. They are bolted through the king-posts, and notched to receive the sides of the latter. The upper crown pieces, six in each rib, are bolted to the crowns of the several trusses by four bolts to each truss. These bolts also pass through the filling in pieces, which are inserted to form the proper curvature for the covering planks to rest upon.

The covering planks are composed of a double thickness. Those which rest immediately on the filling in pieces are 12 inches wide by 6 inches thick, and are placed 6 inches apart. Above these are battens 9 inches wide and 3 inches thick, and the several courses of the arch-stones break joint with the battens, as shown in the Plate.

STAINES BRIDGE.

Plate 53.

This bridge was erected over the River Thames, at Staines, from the designs and under the direction of George Rennie, Esq., Civil Engineer.

It consists of three arches, namely, one of 74 feet span and two of

66 feet each. These arches are segments of circles, having a rise or versed sine equal to one-eighth of the span.

PIERS AND ABUTMENTS.

The piers and abutments are all founded on piles. These are 18 inches in diameter, and are driven down at distances of 5 feet apart from centre to centre over the whole space to be occupied by the masonry. To the heads of these main piles are bolted transverse bearing logs, 18 inches square, and these again are covered by close planking, 6 inches in depth, laid lengthwise of the piers and abutments, that is, in a direction at right angles to the line of the road-way. Upon this planking is laid the lower course of the masonry at the level of 21 feet 6 inches below the springing of the arches. From this level the abutments are carried up to the impost course with a batter on the face of 1 in 10. The piers have each four courses of footings, each course having a set-off of 3 inches all round. The footings are 6 feet in depth, so that the surface of the upper course of these is 15 feet 6 inches below the springing of the arches. The thickness of the piers is 10 feet immediately above the footings, and this thickness diminishes to 8 feet 6 inches at the under side of the impost, from which it will be seen that they are built with a batter equal to one-twentieth of their vertical height. The piers are built with circular cutwaters which are carried up with the same batter as the body of the pier. The extreme length of the piers from point to point of the cutwaters is 43 feet 9 inches at the under side of the impost. All round the lower course of the piers and abutments is driven a row of sheeting piles with a waling bolted to it on each side. The sheeting piles of the abutments are 11 feet in length by 14 inches by 9 inches; those of the piers 12 feet 6 inches in length, with the same sectional dimensions as those for the abutments. Each of the walings is 18 inches in depth by 9 inches, and all firmly secured to the piles by through bolts at every alternate pile.

The abutments are 16½ feet in thickness at the foundation, diminishing to 15 at the springing. Each abutment terminates in a pilaster 11 feet in width, and projecting on the river side 7 feet, and on the land side 1 foot 6 inches beyond the line of the front. The side of the abutments terminates against the pilaster in a curve corresponding with the cutwaters of the piers, as will be seen by reference to the plan. There is a peculiarity in the abutments of this bridge which is worthy of notice, namely, that the courses, instead of being horizontal, radiate to a succession of points situate in the versed sine of the arch produced. Thus the lower course radiates to a point about 27 feet above the centre from which the curve of

the arch is described, and each succeeding course radiates to a point successively 2 feet below the last, till at length the uppermost course of the abutment (that is, the springing course) coincides with the arch-stones in radiating to the same point, namely, the centre of the curve. This radiation does not commence exactly at the face of the abutment, as the bottom courses for about 4 feet in breadth of the abutment have horizontal beds. The extent of horizontal bed diminishes, however, according to the height of the abutment, till at the springing course only 9 inches of its under bed is horizontal, and the whole of its upper bed radiates, as already described, to the centre of the arch.

WING WALLS.

These are founded on piles 12 feet in length, covered by cross sleepers and planking similar to that in the foundations of the piers and abutments. The lower course of the wing walls is laid 11 feet below the surface of the ground and 17 feet below the springing of the arches. These walls have one course of footings, 4 feet 9 inches in breadth, and above this the wall is carried up with a thickness at base of 4 feet 6 inches, diminishing to 3 feet at the string course. The wing walls terminate in upright pilasters corresponding with those in front of the abutments. A small arch of 10 feet span is thrown in the wing walls, and an opening continued through between the pilasters, as shown in the elevation.

ARCHES.

The breadth of the arches across the bridge from outside to outside is 34 feet. The thickness of the arch-stones at the springing is 5 feet 6 inches, diminishing on each side to 2 feet 4 inches at the crown. The exterior arch-stones are cut not in a curved form but with square heads, as shown in the elevation, in order to form proper joints with the horizontal courses of the spandrils.

SPANDRILS.

The outside spandril walls are 3 feet in thickness, and there are six interior spandrils, each 18 inches thick, built at equal distances apart, and parallel with the outside walls. In the whole of the outside masonry, from the springing of the arches to the cornice, the edges of all the stones are chamfered, as indicated by the double lines showing the joints in the elevation.

PARAPET.

The parapet is without coping, and is 4 feet in height, part of which consists of a basement 18 inches square, above which the wall diminishes to 15 inches in thickness.

ROAD-WAY.

The breadth between the parapets is 30 feet, and comprises a carriage-road 21 feet wide, with a foot-path of 4½ feet on each side. Beyond the abutments the road-way becomes wider, as shown on the plan. The approaches are raised on each side about 17 feet above the surface of the ground. They consist partly of brick arches, as shown in the elevation, and partly of earthen embankment.

WELLESLEY BRIDGE, LIMERICK.

Plates 54 and 55.

This is a bridge of five equal arches, each of 70 feet span, erected by the late Alexander Nimmo, Esq. The length of the bridge from end to end of the parapet walls is 410 feet, and the breadth of road-way between the parapets 41 feet.

PIERS AND ABUTMENTS.

The two centre piers have each three courses of footings, each course being 18 inches in thickness. The other two piers have each two 18 inches courses of footings. These courses in all the piers diminish all round by offsets of 6 inches, and above the upper course of footings each pier is 12 feet 6 inches in breadth by 55 feet 6 inches in length from point to point. The piers are carried up 12 feet above the footings, that is, to the springing of the elliptical arches, with a curved batter of 15 inches all round in the height of 12 feet. Their dimensions, therefore, at the springing of the elliptical arches are 10 feet in breadth by 53 feet in length. Up to this height the piers above the footings are built of squared masonry on the outside, filled in with coursed rubble, as shown in the longitudinal section, Plate 55. The courses of ashlar in the piers average 2 feet in thickness.

Each abutment has one course of footings, which also extends beneath the short wings at the ends. Above this course the abutment proper is 47 feet in length by 16 feet 6 inches in thickness. At each end of this length the thickness diminishes by a curve to 7 feet 6 inches, which is the thickness of the wings. The front or inside of each abutment is carried up with the same curved batter as the piers. The outer side of the abutment is carried up perpendicular with two sets-off, as shown in the elevation, Plate 54. Each abutment has four counterforts 6 feet by 4 feet, and each wing has two counterforts 4 feet square.

Above the springing of the elliptical arches the piers and abutments are carried up vertical to the height of 19 feet 4 inches above the footings, that is, to the under side of the springing course for the circular arches. The springing course is 20 inches in depth, and is surmounted over the ends of the pier fronts by an ornamental cap of great elegance and simplicity.

The arches in this bridge are of that complex form adopted by Perronet in the Pont Neuilly, and by Telford in the Gloucester Bridge. They consist here of circular segments, with a rise or versed sine of only 8½ feet, swelling into the elliptical arches which occupy the central line of the bridge. They present, therefore, that bell-mouthed appearance which has been so much admired by several of our greatest engineers, and which, while it certainly affords no greater capacity for the escape of water than an entire arch coinciding with the interior curve, is yet singularly elegant in appearance, and is undoubtedly much stronger than a plain segmental arch of the flat curvature in the design before us.

The outer spandrils are 2 feet in thickness. They are built of dressed stone, one course of which projects on the inner side, as shown in the transverse section, Plate 55, in order to support the covering of flag-stones, which will presently be described. The space between the spandrils is filled up solid with coursed rubble to the height of 12½ feet above the springing of the elliptical arches. Above this height are built six interior walls, at distances of 6 feet apart from centre to centre. These walls are parallel with the outer spandrils, and are built of picked and coursed rubble to the height of 6 feet 6 inches, which level corresponds with the top of the projecting course in the outer spandrils. At this level a course of flag-stones is laid on the tops of all these interior spandrils, forming a platform to receive the road-metalling, and leaving the spaces hollow between the several lines of spandril walls. The cornice is composed of a bed-mold and fascia course, shown in Plate 55 upon an enlarged scale. The parapet consists of a plinth, ballister, and capping (shown in the same Plate), with two pilasters over each arch, and a double pilaster and panel over each pier. The breadth of carriage-way is 28 feet, and that of the path on each side 6 feet 6 inches. Paved drains are formed on each side of the carriage-way, as shown in the transverse section, Plate 55.

Although this bridge is not nearly so large as many similar works in this country, and in point of magnitude is even small in comparison with the noble structures over the Thames, it would perhaps be difficult

to fix upon a design for a stone bridge in any part of the world which is more worthy of admiration than Mr. Nimmo's chaste and elegant bridge at Limerick. The most exquisite architectural beauty is here joined with the most complete application of those scientific principles which Mr. Nimmo's accomplished mind so well and so successfully brought to bear in all his works. Just and true in all its proportions, singularly beautiful in the eyes of every spectator, there is here no redundancy of trifling ornament, no spurious gewgaws, no violation of architectural discipline, to offend the taste of the most fastidious critic; and with all these merits there is such a nice adaptation and adjustment of all the parts in respect of strength and resistance, that the practical man who looks at the most elegant design as a piece of mechanism, is no less satisfied than every one else with the surpassing merit of this elegant bridge.

We feel proud and happy that, amongst the works of many other engineers whose labours it has fallen to our lot to describe in these pages, there is more than one which illustrates the skill and talents of Mr. Nimmo. This gratification is increased by the reflection that the character of this great man is not sufficiently known and estimated by the rising engineers. It will be far from the object of the present descriptions to verge towards any thing in the shape of biography. We shall therefore conclude these observations by referring the young engineer to the various works executed by Mr. Nimmo, in Ireland, for some very fine examples of skill, taste, and engineering judgment. As for the Wellesley Bridge, in particular, it is probable that no higher compliment could be paid than to pronounce it in every way worthy of the fame and genius of Alexander Nimmo.

BRIDGE OF JENA.

Plates 56 and 57.

This bridge is constructed across the Seine, at Paris, in front of the École Militaire, according to the design and under the direction of M. Lamande, engineer-in-chief of the corps of bridges and highways.

An edict for the construction of a bridge in front of the École Militaire was passed on the 27th March, 1806; and it was at first proposed that the bridge should be of iron, with piers and abutments of masonry; but in conformity with a new decision, the arches of the bridge, as well as the other parts, have been constructed of stone instead of iron.

An imperial decree, issued at Warsaw on the 13th January, 1807, declares that this bridge, then building in front of the École Militaire, shall be called the Bridge of Jena, in commemoration of the victory achieved by the French arms at that place.

The bridge consists of five equal arches, each of 92 feet span, and the breadth of road-way between the parapets is 42 feet 6 inches.

THE ABUTMENTS.

These are founded upon piles driven down over the whole area, at intervals of about 4 feet. On the left bank of the river the spaces between the pile heads were filled in with rubble, grouted with lime mortar, and covered over with a platform, on which rests the masonry of the abutment. The abutment on the right bank was constructed in a caisson which rests upon a pavement of hard stone, set by hand between the heads of the piles. Both the abutments are built of squared masonry of hard stone. They are 45 feet in thickness from the face of dressed masonry to the outside of the rubble backing, and the face is carried up vertically to the height of 36 feet, measuring from the foundation to the under side of the cornice or string course. At the ends of the abutments commence the two walls of the quay, which are built along the bank of the river. The face of the abutment is 3 feet 6 inches in advance of the face of these walls at their foundation, and as these have a batter of 1 in 10, the face of the abutment is 7 feet in advance of them at the level of the under side of the cornice. The dimensions of the abutments will be seen from the elevation and from the plan in Plate 57.

PIERS.

The four piers are each 11 feet in thickness at the foundation above the footings, and 10 feet in thickness at the under side of the impost or springing course. They are carried up solid, with semicircular ends, as shown in the plan, Plate 57. The first pier from the left-hand side of the river rests upon two courses of footings which are founded upon piling, covered by a wooden platform. The other piers were founded in caissons, of which the bottoms rest upon piling driven down to the level of 5 feet below the top of the footings. The spaces between the pile heads were filled with rubble stones, grouted with a coarse concrete of quicklime, sand, and gravel. Sheeting piles are driven all round them, and the outside of these is surrounded by a rough protection of large rubble stones, thrown in to prevent the foundations from being undermined. On the bottom of each caisson rest three courses of squared stone, forming the footings or foundation courses on which the piers

are carried up. The form of the springing course, with its fillet and projecting impost, and that of the cutwater caps, will be seen from the transverse section and elevation in Plate 57.

ARCHES.

The five arches are all of the same size, namely, 91 feet 9 inches in span, with a versed sine of 10 feet 9 inches, and the curve is a segment of a circle whose radius is 102 feet. The springing of the arches is 20 feet above the surface of the upper course of footings. The depth of arch-stones at the key-stone is 5 feet. Each arch is composed of 67 voussoirs, and the whole exterior of each arch is covered with a bed of cement and gravel, 7 inches in thickness.

ENTABLATURE.

The stone cornice is 3 feet in depth, and is shown in detail by the transverse section and enlarged elevation in Plate 57. M. Lamande has borrowed the idea of this form of cornice from the Temple of Mars at Rome, under the impression that it would greatly contribute to the decoration of this monument.

PARAPETS.

The parapets are 3 feet 2 inches in height, composed of a single plain course of stones set on edge, and terminated at the ends by four pedestals.

FOOT-PATHS AND CARRIAGE-WAY.

The foot-paths are paved with squared stones and separated from the carriage-way by curb-stones of hard and durable material. The carriage-way is formed of paving stones bedded upon a thickness of 7 inches of gravel. The surface is formed of the curve shown in the transverse section, and the drainage and escape of water are provided for by pipes.

TOWING-PATH.

The towing-path is carried under the first arch on the right bank of the river. Its surface is raised 12 feet above the level of the top of the footings, and the retaining wall in front is built of dressed ashlar. The wall has two courses of footings, and was founded in a caisson which rests upon piling, as shown in the elevation, Plate 56. In passing under the arch of the bridge, the towing-path sweeps inward, as shown in the plan, Plate 57; and on each side of the bridge the water-way gains the additional width given by the set-off of the quay wall, close in front of which is the towing-path, as shown by the plan last referred to. The spaces between the pile heads in the foundation of the towing-path wall are filled with rubble stones, the same as in the foundations of the piers. The thickness of the wall immediately above the footings is 7 feet 6 inches, and

the face is carried up with a batter of 1 in 10. The foot of the wall is defended from injury by sheet piling driven all round it and levelled off at the same height as the piles of the foundation which support the bottom of the caisson, and is further protected by large rubble stones thrown in round the sheet piling.

THE DEVIL'S BRIDGE, OVER THE SERCHIO, NEAR LUCCA, IN ITALY.

Plate 58.

This structure, erected about the year 1000, is the only one now remaining of the numerous bridges which once spanned the wild and turbulent Serchio. It consists of one large arch of 120½ feet in span, one land arch on the left-hand side of 17½ feet span, and three arches on the right-hand side, the first from the main arch having a span of 46 feet 10 inches, the second of 33 feet, and the third of 27 feet 10 inches. The large arch is semicircular, and springs at once from the rock which forms the bed of the river. The other arches are also semicircular, but they spring from a higher level than the main arch, as shown in the elevation. The quoins of the smaller arches and all the voussoirs of the large arch are of dressed stone. The piers, spandrils, and every other part of the bridge, are of rubble. The courses of stone in the large arch vary from 8 inches to 21 inches deep, but there are very few of the latter depth.

The surface of the rock was dressed to a somewhat uniform and even face for the foundation of the large pier, but all the other piers are founded upon the natural surface, however irregular.

Mr. R. Townshend, C.E., to whom we are indebted for most of the particulars respecting this bridge, states that the principal kinds of stone used in its construction are a blue limestone and a variety of sandstone. When Mr. Townshend visited the bridge in November, 1838, it had lately undergone some repairs, principally about the parapets, coping, wings, and road-way, and he adds that the structure promises to last for many years to come.

It will scarcely be necessary to observe that this bridge is not presented as a specimen worthy of being imitated in these days, when every element of luxury and convenience enters so essentially into the composition of a bridge design. The bold rude arch which spans the Italian river rises far above the limits demanded for head room by any flood which could ever occur, while the small arches which flank it spring from so low

a level as to impose on the road-way an inclination almost too steep for the passage of wheeled carriages. When to this we add, that in consequence of perpendicular rocks several hundred feet high on each side of the river, the road-way, which is only 9 feet wide, turns very abruptly at the wings, it will at once be seen how remarkably all modern ideas of convenience and adaptation have been violated. Indeed, whatever purpose the bridge may have originally been intended and used for, Mr. Townshend observes that in consequence of its steepness, narrowness, and the angles formed in the road-way at the wing walls, it is at present impassable for vehicles.

Notwithstanding its great defects, however, there is much to admire in this relic of ancient architecture, which, built of the rudest materials, and presenting as it were a mere perforated wall of only 12 feet in thickness to the floods of a rapid and impetuous river, has yet withstood its fury for more than eight centuries. During the dry season of summer, the water of the river, being very low, as shown in the Plate, finds a quiet passage through one side of the main arch; but when swollen by rains and by numerous mountain torrents, the floods of the Serchio assume an immense volume which is borne along with great velocity. These floods commonly reach above one or more of the smaller arches, and deluging the road-way, of course render the bridge quite impassable. Mr. Townshend observes that one can scarcely think it possible, while standing on the crown of the large arch, that a structure so slight could withstand the rush of a head of water upwards of 30 feet deep, sweeping through the arches at the rate of 8 miles an hour.

It will be seen from the plan that a peculiar raking form has been given on the up-stream side to the cutwaters of the piers. This form has been adopted in order to divert and guide the greater part of the flood through the openings of the arches. The direction of the current is shown by the arrow on the plan, from which it appears that this singular form of cutwater is the best adapted to cleave the current and to suffer its passage with the least possible resistance. It is remarkable that except in one instance the piers are provided with no corresponding projection on the lower side of the bridge. We should not omit to notice that this bridge undoubtedly owes much of its stability to the quality of its mortar, an excellence for which Italy, above all other countries, has never ceased to be very highly celebrated.

BRIDGE ACROSS THE RIVER FORTH, AT STIRLING.

Plates 59 and 60.

This is a stone bridge of five arches, erected by Robert Stevenson, Esq., of Edinburgh. The centre arch is 60 feet in span, two others are 58 feet each, and the remaining two are 53 feet 6 inches. The whole length of the bridge from end to end of the parapet walls is 424 feet, and the breadth between the parapets is 32 feet 10 inches.

All the piers, abutments, and wing walls are founded on piles, which are 18 feet in length and 9 inches square. These piles are driven at distances of 4 feet apart from centre to centre, and are shod and hooped with iron, as described for the piling of Hutcheson Bridge, vol. i. p. 106. A waling of beech timber 12 inches in depth is fixed all round the outer rows of bearing piles, and bolted to each pile with two screw bolts, as described for Hutcheson Bridge. Transverse stretchers of dimensions similar to those of the waling pieces are bolted to the inner rows of bearing piles. The sills or sleepers resting on the tops of the bearing piles, and the close planking which completes the timber foundation, are all precisely similar to that specified for Hutcheson Bridge. All the piers and abutments have five courses of footings. In the abutments the lower course of footings is 16 feet 9 inches thick, and the thickness or breadth of each course diminishes by an offset of 6 inches on each side. The depth of the whole five courses is 7 feet. Above the footings the abutments have a basement 2 feet 8 inches in depth and 12 feet 3 inches in breadth. The piers have a basement of the same depth, and 10 feet 2 inches in breadth. The thickness of the abutments above the basement is 12 feet, and from this level to the under side of the impost they are carried up with a slight batter, which diminishes the thickness at that height to 11 feet 8 inches. The piers are 9 feet 8 inches thick above the basement, and batter to 9 feet at the impost. The piers and abutments are built of solid squared ashlar masonry. The wing walls are founded in steps at the several depths shown in the longitudinal section, Plate 60. They form a continuation of the masonry of the abutments, and are connected with the latter at the back by a horizontal counter arch. This arch is of an elliptical form, and is carried up in regular courses composed of header and stretcher alternately to the top of the springing course. The dimensions of the stones and courses for the abutments, piers, and

wing walls are similar to those of Hutcheson Bridge, in the specification of which, already referred to, these are particularly described.

The spans and versed sines of the several arches are figured on the longitudinal section. The intrados or inner circle of all the arches is struck with a radius of 40 feet, and the extrados or outer circle with a radius of 45 feet. The depth of arch-stones at the crown of the arch is 2 feet 9 inches, and this depth increases regularly to the springing, where they measure 3 feet 6 inches in depth. The springers for all the arches are 4 feet 6 inches in breadth, 3 feet 3 inches in thickness, and not less than 2½ feet in length. The breadth of the two lines of springers on each pier covers the whole top of the pier. Resting upon the springing course, a cross wall is carried up over each pier. These cross walls are 3 feet in thickness; they extend entirely across the bridge between the spandril walls, and are carried up perpendicular to the several heights marked on the longitudinal section. A cross wall is also carried up over the springing course of the abutments. These cross walls are 8 feet 6 inches in thickness at the level of the springing, and they diminish by two offsets to 6 feet 6 inches at top. Along the whole length over the abutments, piers, and arches, are built seven interior spandrils, which abut at each end against the counter arches of the abutment, and divide the whole breadth across the bridge into eight compartments. These walls are 2 feet in thickness at bottom, and 1 foot and ½ at top. They are bonded into the cross walls already described, and on the top of all these spandril walls is laid a coping of hammer-dressed flag-stones, 18 inches broad and 7 inches thick. Upon this coping is laid a surface of paving flag-stones, covering the whole space between the outer spandril walls. These paving stones are 7 inches in thickness; they are laid with a sufficient bearing upon the spandril and cross walls, and no stone contains less than 8 square feet of surface. Over this pavement, along the whole length of the bridge, is a clay puddle 12 inches in thickness, and this is covered by a stratum of gravel for the road-metalling, as particularly described in the specification for Hutcheson Bridge. Spaces of 6 inches square are left in the interior spandrils and cross walls, to admit a free circulation of air through the interior of the bridge.

The outer spandril walls are 3 feet in thickness at the level of the springing course, and 2½ feet in thickness at the top. These walls are carried up with radiating courses in continuation of the courses in the arches.

In front of each abutment is a projecting pilaster, 9 feet 6 inches in width.

The pilasters over the piers are each 9 feet in width at the springing course, and 7 feet at the under side of the cornice. All these pilasters, as well as those of the wing walls, are continued above the cornice by corresponding pilasters in the parapet walls. The frieze under the cornice is 1 foot 6 inches in depth, and projects 2 inches over the face of the spandril walls. The stones of this course are not less in length than 3½ feet, nor less in breadth of bed than 14 inches. The exterior surface of these stones, and of all the masonry in the bridge up to this height, is of rusticated work, and all the joints are chamfered.

The cornice is 14 inches in depth, composed of stones not less than 3½ feet in length, and projecting 15½ inches over the face of the frieze. The stones of the cornice are dressed and set in all respects as described in the specification for Hutcheson Bridge. The parapet walls consist of a basement, dado, and coping. The former is 14½ inches in thickness, composed of stones not less than 4 feet in length. The dado is 12 inches in thickness, and the coping 14 inches.

For a particular description of the parapet walls, the road-way, lamp irons, &c., see specification of Hutcheson Bridge.

The piers and abutments of Stirling Bridge were built of hard whinstone, a provincial name given to the volcanic rock called green-stone by geologists. It was originally intended to build the rest of the bridge of free-stone, but as the work advanced it was determined to employ green-stone for the whole elevation. The materials were procured from quarries in the neighbourhood of Stirling, and from the North Queensferry on the Forth. The erection of the bridge was contracted for by Mr. Kenneth Matheson, and its cost amounted to about £17,000.

TIMBER BRIDGES, &c. ON THE UTICA AND SYRACUSE RAILROAD,

IN THE UNITED STATES.

PLATES 61, 62, 63, 63[a], 64, 64[a], 65, 66, 67, 67[a], 68, 69, 70, 70[a], 70[b], 70[c], 70[d], 70[e], 70[f], AND 70[g].

(See Isherwood's Practical Description, vol. II. pp. xi. to lxxiv.)

BRIDGE OVER THE RIVER ADDA, AT TREZZO, MILAN.

Plate 71.

The principal particulars which are known respecting this remarkable ruin are contained in a note which will be found on the Plate. The bridge was situate close to the castle of Trezzo, which is now also in ruins, and which was formerly the residence of the Duke of Milan, mentioned in the note referred to. The bridge was built as a communication between the castle and the opposite bank of the river, and is supposed to have been destroyed about 120 years after its erection. The parts of the arch which remain in position on each side afford the means of restoring the line of the curve on a drawing, and show the arch to have been one of such immense size, that the absence of information respecting the whole mode of its construction is much to be regretted.

ELY BRIDGE, OVER THE RIVER OUSE, AT ELY, CAMBRIDGESHIRE.

Plates 72, 73, 74, and 75.

This is an iron bridge constructed by the Butterley Company for the Dean and Chapter of Ely, from the designs of Joseph Glynn, Esq., F.R.S. The bridge is erected within a short distance of the cathedral, an edifice comprising almost every variety of style in ecclesiastical architecture, and the approach to it is lined by many quaint and curious buildings. The design seems intended to impart to the structure a character in unison with the localities of this ancient city of the fens. The bridge is 136 feet in length from end to end of the wing walls, and is built in an oblique direction to the line of the road-way. The two arches are each 42½ feet in span, measured on the skew line. Each arch is composed of four ribs, supporting a road-way 13 feet 6 inches wide. The pier rests upon piles, the heads of which are covered by a double thickness of 3-inch planking. Above this are four courses of footings, each a foot in depth, and the two upper courses are diminished by offsets of 6 inches all round their sides and ends. The plinth of the pier is built upright, 5 feet in height, and at this level there is a set-off of 6 inches. Above the plinth the height of the pier up to the springing is 6 feet. It is built of squared masonry, and carried upright with a thickness of 5 feet 3 inches, as shown in the plan, Plate 72. The abutments are founded at a depth of 7½ feet below the level of water

in the river: they rest upon piling and planking similar to that of the pier. Each abutment has three courses of footings, the two upper courses having offsets of 6 inches all round, except at the back, where they are carried up perpendicular. The plinth of the abutments is the same height as that of the pier, and terminates at the same level. In front of each abutment is a pilaster 4 feet 6 inches wide up to the top of the plinth, and 4 feet wide above that height. These pilasters project 2 feet beyond the face line of the bridge, and correspond with the small pilasters in the parapet walls. The wing walls are founded 2 feet 9 inches below the top of the plinth; and the set-off at the top of the abutment plinth is continued along the face of the wing walls. At its junction with the abutment each wing wall is 10 feet in height from the top of the plinth to the under side of the string course; and at the extremity of each wing wall its height between the same two points is 8 feet, the difference of level being occasioned by the slope of the road-way. The wing walls are built in a curved form, with pilasters at the ends, as shown in the plan, Plate 72. The stone parapet over the wing walls is quite plain, with a coping 12 inches in thickness.

Plate 73 contains an enlarged elevation of part of a rib, and plans of the abutment and pier plates on which the ribs rest. The abutment plate is 14 feet 9 inches long, 3 feet wide on the square, and $1\frac{1}{2}$ inch in thickness, with a raised flange of 2 inches in height all round it. The pier plate is the same thickness as those of the abutments, and is surrounded by a similar raised flange. These flanges are notched at the proper intervals to receive the ribs of the arches. The abutment plate is formed with three hollows, which serve to diminish the quantity of iron in the casting. These hollows are 2 feet long and a foot wide on the square; their figure is rhomboidal, with the corners rounded off. The pier plate has also three rhomboidal hollows, 2 feet 8 inches long and 3 feet 3 inches wide on the square.

Each of the ribs is an inch and $\frac{3}{4}$ in thickness, and each is provided with a flange on one side to receive the bolts for securing it to the pier and abutment plates. This flange abuts against projecting pieces cast on the plates, and is wedged tightly into its place by a key driven between the rib and a projecting boss, cast on the surface of the plates, as shown in the plan.

Each rib is 96 feet in length by $1\frac{3}{4}$ inches thick, and is formed in such a manner as to present the appearance of two separate arches; its under side forming two very flat curves, the ends of which are turned down, so

as to prevent any lateral pressure from acting against the abutments. The ribs are cast in three pieces, connected by means of a scarfed joint, and secured laterally with 1-inch screw bolts, placed 7 inches apart from centre to centre, and longitudinally by strong dowels and keys: see the elevation of the joint, Plate 74.

The section of each rib presents the form of a cross, the internal ribs having two square projections of 1 inch cast upon them, to support the covering plates and give additional strength to the rib; the external ribs have a similar projection on the inside for the same purpose, with a corresponding rounded one on the outside.

The coved cornice extending in front of each outside rib, as well as the plinth or socket of the parapet railings, and the spandril plate between these, form one piece of casting, which is secured to the ribs by countersunk rivets. The cornice is hollow, and is formed of ¾-inch cast iron, resting at the bottom upon the circular projection of the rib; this cornice is 7 inches deep, and projects 5½ inches in front of the spandril plate; the railing plinth, which is 4 inches in depth, is formed with a small elliptical-shaped top, 4 inches in width. The part of each rib which rests upon and adjoins the pier is cast with hollows, as shown in the elevation, and ornamented with mouldings to correspond with the general design. The end of each rib where it adjoins the abutment is ornamented with two raised shields, one bearing the arms of the See of Ely, parted with those of Dr. Sparke, the late bishop, and the other the arms of the Chapter of the Cathedral, parted with those of the late dean, Dr. Wood, the venerable master of St. John's College, Cambridge: there is also a raised inscription in old English characters, stating that the bridge was cast by the Butterley Company for the Dean and Chapter of Ely, at Butterley Iron Works, Derbyshire, Anno Domini MDCCCXXXIII.; and there is a moulded tablet-work over the pier to correspond with the general design, as seen by the elevation, Plate 72.

The balustrade, extending from one abutment pilaster to the other, is 3 feet in height, and consists of a series of 1½-inch shafts, placed 6 inches apart from centre to centre, each presenting the appearance of a Gothic mullion: the plan and section of these shafts are shown very clearly in Plate 75; they are united to each other at top and bottom by a Gothic scroll arch, which not only gives stability to the railing, but beautifully harmonizes with the Gothic style of the adjacent cathedral.

The railing is strengthened and relieved by the interposition of a larger shaft or principal of the same pattern as the smaller ones. Every tenth

shaft is of the larger size, and by these latter shafts the railing is keyed to the upper part of the ribs.

The whole is surmounted by a neat oval-shaped hand-railing, shown on an enlarged scale in Plate 75.

The road plates shown in Plate 75 are of ¾-inch cast iron; they are 3 feet 9 inches wide, and rest upon the bearing parts of the ribs; four flanges, with convex tops, are cast upon each plate, ¾ inch wide, 4 inches deep in the centre, and 10 inches apart. The plates are also provided with edge flanges 3 inches deep by ¾ inch wide; these are bolted to the ribs at each extremity by four ½-inch screw bolts, so that they serve the purpose of lateral stays to the ribs.

The plates are brought together lengthwise without any flange or other fastening, with a simple butt joint, and are covered by the road-metalling.

The bridge has embanked approaches, rising with an easy inclination on each side to the crown of the arch.

HADDLESEY BRIDGE, YORKSHIRE.

BY JOSEPH GLYNN, ESQ., F.R.S.

PLATES 76 TO 83.

This is a bridge built by the Butterley Company over the River Aire, at Haddlesey, on the new line of the Great North Road from Doncaster to Selby.

It is a cast iron bridge of three arches; the span of the centre arch being 70 feet, with 7 feet rise, and that of the side arches each 50 feet, with 6 feet rise; its extreme length is 252 feet, and the width of the carriage-way 24 feet.

The piers and abutments are founded upon piles 14 feet in length by 1 foot square, placed 2 feet 6 inches apart from centre to centre. They are covered by a double thickness of 3-inch planking, this forming a platform upon which the foundations are laid, 12 feet below the surface of the water; the foundations consist of two 18-inch courses of stone footings, the upper one having an offset of 6 inches all round.

The piers measure at the under side of the footings 11 feet wide by 41 feet in length.

Above the footings the ends of the piers are worked off on the plan to the figure of a Gothic arch, forming a cutwater at each end; the piers

measure 39 feet from point to point of these cutwaters, and are carried upright to the height of 10 feet above the footings, where two sets-off occur, which reduce the width of the pier to 7 feet. At the height of $14\frac{1}{2}$ feet above the planking the cutwaters are surmounted by a mitred coping 5 feet high and 7 feet wide.

The shafts of the piers are continued up to the springing line, with plain squared sides and ends of masonry, without any batter.

Plate 80 contains a section of the abutments taken at the centre line. These, like the piers, are built of squared stone masonry, and their dimensions are fully detailed in the Plate.

The abutments are worked into the wing walls, and are strengthened at the back by a counter arch of a semicircular form; the intrados of this arch is worked with flush face to a radius of 8 feet 6 inches. It is built of heavy ashlar, close jointed, and fitted in the most substantial manner; its extrados is also worked with vertical joints and beds, and is squared to suit the abutting courses of the wing walls and abutments.

This counter arch is intended in an effectual manner to resist the thrust of the arch: see Plate 82.

The wing walls, founded upon footings, are formed with a slight curve outwards, and stepped up the slope, with offsets, as shown in the Plate. They are terminated by a small pilaster 6 feet high, founded 2 feet below the surface of road.

In Plate 80 is shown a section of the stone parapet over the abutments and wing walls; each parapet is 30 feet in length, and is surmounted by a plain coping 9 inches in thickness, with its upper edge washed off to the depth of 3 inches.

The horizontal springing line of the central arch is 9 feet 6 inches above the level of the water. That of the two side arches inclines in a downward direction from the middle of the bridge, so as to be at the abutments 8 feet above the water level, and to correspond with the embanked approaches, and consequent curve given to the road-way and string course.

Plate 76 comprises a plan of part of the iron pier plates, which are placed upon the piers to support the ends of the ribs; this pier plate is 6 feet 4 inches in width, and is firmly secured to the masonry of the pier by $1\frac{1}{2}$-inch screw bolts; six rectangular hollows, each 2 feet 4 inches long by 3 feet wide, having the corners rounded off, are cast in each pier plate, one between each pair of ribs, leaving 1 foot 6 inches of plate between the spaces.

The abutment plates are placed in a direction radiating to the centres of the side arches; they are cast with three small hollows, the same length as those in the pier plates.

Plate 78 shows the strong cast iron frame-work, or standard, which is fixed over the pier, and firmly secured by bolts and keys to the pier plate; it also shows the bolt holes by which the ribs are attached to it; these are 1½ inch square, and are placed 9 inches apart from centre to centre on one side, and about 1 foot 8 inches apart on the other; the standard is 5 feet in width, and is overlapped on the top by the cast iron cornice.

The section of one of the diagonal braces of the standard is also given in this Plate, and the elevation shows that the standard has triangular openings with rounded corners. The mouldings and general outline of the standard are of a plain Grecian character, which corresponds with the general style of the fabric.

Each arch consists of seven ribs; those of the centre arch are cast in three pieces, connected by lapped joints 21 inches long, and made fast by six screw bolts. The ribs of the side arches are each cast in two pieces, connected at the centre of the arch by a lapped joint 18 inches long, with four bolts passing through and secured by nuts: these connexions are seen in Plate 77.

This Plate also represents the plan and elevation of one rib of the centre arch, and one for the side arches, from which will be seen their general dimensions, and those of the four hollows cast in them.

Plate 78 represents an enlarged elevation of the outside rib of the centre arch, and shows a section across the bottom part of the rib, from which it appears that the thickness of the plate of the rib is 1½ inch, that the bottom flange is 5½ inches wide, and that each hollow is bounded by an oval-shaped moulding 4 inches in width: the small transverse section in Plate 77 shows the form of the middle parts of an inside and outside rib for the centre arch.

The inside ribs are cast with a projection 3 inches wide on each side to support the road-way plates, and the external rib has a projection of the same kind on the inside only. The bottom flanges are of course 5½ inches wide, similar to that shown on a larger scale in Plate 78.

The bridge is strongly braced transversely and diagonally, as shown in Plate 77; the transverse braces consist of distance tubes alternately 4 inches and 6 inches in diameter, and 1½ inch thick; these tubes are placed at the intersections of the diagonal braces, and are formed with flanges which abut against the ribs.

Through the line of tubes which is fixed at the centre of each arch passes a wrought iron tension bar 1 inch square, which extends entirely across all the ribs; the diagonal braces are attached to the ribs by flanges.

The plan in Plate 77 shows the position of these braces in one direction, and their position on a vertical plane is shown by the dotted lines on the elevation in the same Plate.

Plate 79 is a transverse section showing on a large scale a side elevation of the standard on the pier; the small hatched figure on one of the braces shows its cross section, which, together with the other hatched sections referring to the top and bottom parts of one of the ribs, will be readily understood. This Plate also shows the hollow moulded cornice, with its flat top projecting 8 inches beyond the plinth of the railing.

The palisade is of the antique water-plant pattern; the plinth is 10 inches deep, and the shafts of the palisade are placed 6½ inches apart from centre to centre; the coping rail is 10 inches wide and 4 inches deep, making the whole height of the balustrade 4 feet; a larger shaft occurs at intervals, and the extremity of this larger shaft is keyed to the plinth, as shown in Plate 82.

A plan of a covering plate is seen in Plate 76; these plates are 4 feet long by 3 feet 9 inches wide, and ¾ inch thick, with an edge flange 3 inches high and ¾ inch thick, cast upon two sides and bolted to the ribs.

Each plate has also three ribs cast upon it in a transverse direction, ¾ inch thick, forming in elevation the segment of a circle. These ribs are 4½ inches high in the centre, 2½ inches at the sides, and 11 inches apart.

There is also an under-lip cast upon each plate for the adjoining one to rest upon. This under-lip is in the form of a half oval, and is shown, full size, in Plate 75.

BRIDGE OVER THE LEA CUT, ON THE LONDON AND BLACKWALL RAILWAY.

Plates 84, 85, and 86.

This bridge crosses the Lea Cut with a single arch of 87 feet span and 16 feet rise; its length is 126 feet, and the width between the balustrades 24 feet.

It was built by Messrs. Grissell and Peto for the Blackwall Railway Company, at a cost, including the permanent way, of £4850.

The foundations consist of a bed of concrete, faced with a brick wall returned round the ends of the abutments.

The whole of the railway being constructed on arches, a small part of those on each side is shown in Plate 84. It will be seen from this Plate, that the arch adjoining the west abutment of the bridge is built in an oblique direction, thus giving to it the form of abutment shown in the plan. The concrete for this abutment is 7 feet 3 inches thick, being flush with the outside of the footings at the ends and back of the abutment.

The face wall in front of the concrete is founded 9 feet 6 inches below high-water mark, as shown on the elevation, Plate 84; it is 34 feet long, 3 feet thick, and 8 feet 3 inches high, being brought to a level with the top of the concrete, and returned round the ends to the length of 7 feet 6 inches on the north side, and 9 feet 6 inches on the south.

The western abutment is raised upon eight courses of footings, having an aggregate depth of 2 feet, with offsets of 2½ inches. Its length at the base of the footings is 30 feet, and its breadth 45 feet at the north end, and 24 feet at the south end.

The pilasters in front of each end of this abutment are 17 feet wide, and project 2 feet from the face. The pilasters are built upright to the under side of the string course, where they are terminated by a frieze which is 2 feet deep, and projects 2 inches. The pilaster is of brick-work to the height of 2 feet above the footings.

This is succeeded by two courses of rustic-dressed ashlar facing, with chamfered joints, the lower course being 2 feet 6 inches deep and 2 feet 6 inches wide, and the upper one 2 feet 3 inches deep and 1 foot 6 inches wide.

The remaining part of the pilaster between the face courses and the frieze is built of bricks, with quoins of ashlar stone, rustic-dressed, and chamfered on the face, placed with an alternate header and stretcher in courses 2 feet thick.

All the stone used in this bridge is from the New Leeds quarry.

The plan of the abutment at the upper side of the footings contains a trapezoidal space, the sides of which are 17 feet 6 inches and 4 feet 6 inches, with a perpendicular of 18 feet. This space is filled with concrete, and carried up in the same form to the top of the abutment, that is, to the height of 13½ feet above the springing of the arch.

The springing course is of ashlar, 5 feet deep by 5 feet wide, and projecting 3 inches in front of the abutment face; its upper bed is made to radiate to the centre of the arch. The remaining part of the abutment is

built of brick-work, laid in lime mortar, and carried upright to the height of 20 feet from the under side of the footings, and finished off level at the top.

The eastern abutment has a concrete foundation, the under side of which is 5 feet 6 inches below high-water level, with a face wall of brick-work, 3 feet thick and 6 feet 6 inches high, brought flush with the upper side of the concrete, and returned 10 feet 6 inches round the north end, and 9 feet 6 inches round the south end; its footings are similar in number and dimensions to those of the other abutment.

The eastern abutment measures at the top of the footings 16 feet 6 inches wide by 27 feet long, and is carried up solid, built internally of brick-work, with two face courses resting upon the footings. The pilaster with its quoins, the frieze, and the springing course, are similar to those described for the western abutment.

It was deemed advisable in the designing of this bridge to extend the span of its arch considerably beyond either margin of the cut, so as to admit of any future increase in its depth without injury to the bridge, and at the same time to avoid the expense of coffer-dams.

The arch measures on the outer rim 3 feet 9 inches in thickness, the internal thickness being 4 feet 1½ inch. The curve forms the segment of a circle, of which the internal radius is 67 feet: it is built of good stock bricks, set entirely in cement, bonded together in each direction. The darker brick-work of the arch, with its cement joints, contrasted with that of the spandrils, which are faced with Malm paviours set in mortar, well defines its outline, and has a somewhat novel effect; it is faced on either side with a key-stone 4 feet thick, projecting 3 inches below the soffit of the arch, with a moulding on the face, projecting 15 inches, and measuring 2 feet 8 inches wide at the soffit.

As during the progress of the work the arch was found at intervals to open in a slight degree on its extrados; after the arch had been keyed in, and previous to striking the centres, thin iron plates were driven into the openings, right across the arch, as far through as possible towards the soffit, and the spaces were also well run in with liquid cement.

This was done to prevent the crushing of the bricks and cement at the soffit, which it was feared would take place when the centres were struck, and the arch brought to its bearings.

The concrete backing of the arch is set to a good current for the drainage, its centre line forming a tangent to the curve in a longitudinal direction, and descending to the level of the concrete in the abutments;

its transverse section is represented in Plate 85: see section through centre of abutment at D D.

The whole surface of the backing is coated with a description of bitumen of English manufacture, which is also used generally along the line, and is found to answer the purpose remarkably well, as it most effectually preserves the brick-work from the wet.

The spandril walls are 2 feet 6 inches in thickness, and are surmounted by the string course, which is 18 inches deep, with blocks 1 foot wide and 1 foot 6 inches apart to support the block cornice above it. This latter is 18 inches deep and 4 feet wide, bevelled off at the top to the depth of 3 inches, and slightly hollowed at the under side of the drip.

The substantial iron railing, forming the parapet of the bridge, is bedded into the cornice, and the several lengths are supported by means of strong ornamental brackets. In addition to these, in order to stiffen and connect the whole together, a concealed wrought iron bar is riveted on the top of the railing throughout its entire length; these rivets pass through the top of the cast iron moulded coping which crowns the whole, and encloses the longitudinal bar mentioned above.

The small lengths of parapet at the ends of the railing, over the abutments, are of stone, 3½ feet thick, and 5 feet high in the middle.

The centering consisted of five ribs, fixed 6 feet 6 inches apart from centre to centre, and bound together by horizontal and diagonal braces: each rib was supported upon four bearing piles, placed 23 feet 6 inches apart, with crown ties, trussed and strengthened by diagonal braces, secured by bolts and strap irons to the filling in pieces. This centering not being of the expensive description usually adopted where so much space is required to be kept clear for the navigation, it was thought necessary to load it in regular courses with the bricks intended for the construction of the arch, so as to test its efficacy, and to bring it well to its bearings before the arch was commenced. The centering itself stood the test remarkably well, but a portion of the piling gave symptoms of sinking below the rest: this was effectually remedied, however, by a judicious employment of timber, so applied as to brace the whole firmly together; and any settlement which afterwards took place was so perfectly regular that no injurious effects resulted.

SUSPENSION BRIDGE UPON MR. DREDGE'S PRINCIPLE, AT BALLOCH FERRY, DUMBARTONSHIRE.

PLATE 87.

(See the Specification, vol. I. p. xliii.)

PERRONET'S DESIGN FOR A BRIDGE AT MELUN.

PLATE 88.

(See Hosking's Preliminary Essay, vol. II. p. 37, &c.)

BRIGHTON CHAIN PIER.

PLATES 89, 90, AND 91.

The chain pier at Brighton was projected by Captain Brown, R.N., and erected by him in the years 1822-3, at a cost of about £30,000. The pier extends into the sea 1134 feet, and the suspension chains on each side of the road-way are supported on four pyramidal iron towers, the distance between which from centre to centre is 255 feet. One of the figures in Plate 89 shows a plan of the double iron tower which forms one of the piers of suspension. The three first of these piers are each founded upon twenty piles, which are driven several feet into the solid chalk, and the fourth pier, being the one farthest from the land, is founded upon one hundred and fifty piles, driven in the form of a T on the plan, and strongly secured by diagonal braces and walings. Extending beyond the towers of this last pier is a platform 80 feet in length by 40 feet in breadth, paved with Purbeck stone, and beneath are galleries and flights of steps for the convenience of landing and embarkation. Each of the towers is bedded on two iron sills or plates, 2 feet wide and an inch and a half thick, securely bolted to the pile heads.

The suspension chains on the land side are carried 54 feet into the cliff, and secured to two large mooring stones and to a massive iron plate more than a ton in weight. These chains, after passing over each of the suspension towers, dip towards the sea, and are firmly secured to the diagonal piles at the pier head. There are four suspension chains on each side of the road-way, and each consists of links 10 feet long

and 6¼ inches in circumference, the weight of a link being about 112 lbs. Each link has a round eye hole at the end of 2 inches diameter, and the connexion of the links is made by two coupling links, pins, and keys, as shown in Plate 89. The iron of the coupling links is 1½ inch deep by 1 inch thick, and the pins are 2 inches diameter.

The four chains on each side of the road-way are suspended in pairs, the upper pair hanging about 2 feet higher than the lower. At each joint of the links forming the chains is a saddle to receive one of the suspending rods. The sketches in Plate 89 explain the form of this saddle, which rests upon and between the coupling links of the suspension chains. The lower part of the saddle is cast with a hollow of 3 inches in length by 1 inch and ¾. This hollow receives the T head of the suspending rod, which is then turned round across the hollow so as to rest firmly in the saddle, without danger of being withdrawn. The top of one of the suspending rods is shown by sketches in Plate 89; the plan of the saddle shows the hollow to receive the head of the rod, and the dotted part across this hollow shows the position of the head when fixed. The lower extremities of the suspending rods are made in the shape of a fork with two flat prongs. Through these prongs are loop holes which receive an iron wedge or cross piece, and this latter supports the longitudinal iron bearing beam which extends on each side of the road-way from tower to tower. The form and dimensions of the suspending rods and the forked part of the same are clearly shown in the Plate. There also will be seen the details of the cross pieces or wedges and of the bearing beam, which it appears was 4 inches in depth by 1 inch in breadth. On this bearing beam rest the girders which support the deck planks of the road-way. These girders are 3½ inches wide by 7½ inches deep at the ends, and 10½ inches in the middle. They are placed 5 feet apart and are covered by the planking, which is 2½ inches in thickness. The ends of these girders support a gutter or channel to carry off the water which runs off the road-way. Details of the fascia and cornice, and of the iron railing which surmounts them, are shown in Plate 89.

The other two Plates (90 and 91) referring to this work are intended to explain the nature of the damage occasioned to the pier by a violent storm which occurred on the 15th October, 1833. Plate 90 shows the two platforms or divisions of the pier which were injured. The one on the right-hand side, which is the second division from the land, had twenty of its suspending rods carried away, and some others bent.

The consequence was that the deck of the pier sank nearly 6 feet on one side, presenting an inclined plane transversely, the appearance of which is faithfully represented in the engraving. The cast iron towers adjoining this division of the pier were also considerably damaged, the sills on which they rest being split so as to cause the iron framing of the towers to lean over on one side. The third division from the land, however, being that on the left-hand side of Plate 90, presented the most complete ruin. No less than forty of the suspending rods were here destroyed on the east side, and about half that number on the other side. The chains in this part of the pier were greatly deranged, as appears from the engraving, and fully three-fourths of the platform and heavy iron railing were completely destroyed. Plate 91 presents another view of this division of the pier, taken from a more oblique position.

It is remarkable that, notwithstanding the violent injury which this storm produced, the light iron bearing bar, with a sectional area of only 4 square inches, was not broken, but held together with a tenacity which afforded strong proof of its excellent quality. The curved form assumed by these bearing bars, which, it will be remembered, supported the road-way beams on each side of the pier, will be best seen from Plate 90, which shows nearly a front view of the pier. The dip of the lower one must have been about 14 feet, as it touched the surface of the sea at high water. It appears from a remark by Mr. Noble, in Plate 89, that the wrought iron-work inside the forks of the suspending rods had considerably exfoliated; and in a letter written by the same architect immediately after the accident, he states that a prodigious exfoliation of the wrought iron-work had taken place, and alludes with apprehension to the small dimensions of the longitudinal bearing beam already mentioned.

We observe that the same system of supporting the road-way on a longitudinal iron bearing bar, suspended in the fork or stirrup of the vertical rods, appears to have been adopted in the Union Chain Bridge over the Tweed, in the Trinity Chain Pier at Newhaven, and indeed in most of Captain Brown's works. At the same time we feel bound to concur with Mr. Noble in apprehending danger from the exfoliation which invariably takes place in wrought iron, and which of course becomes far more serious in its consequences when it happens to a bar of such small dimensions as the one here referred to.

Many conflicting opinions have been entertained as to the cause of the

destruction which forms the subject of these Plates. Some, and these principally appearing to be the friends and supporters of Captain Brown, have ascribed the accident to lightning, thereby intending to show that, however perfect the construction might have been, it was beyond the power of human sagacity to protect it from the calamity by which it was visited. Others have contended that the whole of the mischief done by the storm may be readily traced to the action of the wind upon the platform of the pier. Mr. Noble observes that the damage was sustained at the time of low water, when there was a height of about 50 feet to be acted upon by an extraordinary and violent westerly wind. He supposes that the wind acted vertically upwards, as well as laterally, and that the deep fascia and projecting cornice opposed a resistance which added to the injurious force of the wind. He observes that a rapid undulation was produced in the platform, and that parts of it were at length forced upwards, the suspending rods at the same time rising, displacing the heavy saddles, and deranging the whole of the chains. For our own part, we have no intention of pronouncing any opinion as to the true cause of the accident, and shall therefore merely observe, that the principal objection to Mr. Noble's view of the case is this, that a force sufficiently violent to have forced up *en masse* the platform of the pier would probably, in the first instance, have torn up the planking of the deck, and have left a free passage for the wind upwards. These planks, it is said, were only secured by four or five nails in each; and it is remarkable that, in the hurricane of 1824, shortly after the erection of the pier, this effect actually happened, a great number of the deck planks having been torn up, while the pier remained comparatively uninjured.

When the damaged part of the pier was restored it was found advisable to make the entablature lighter, and to confine its depth to the under side of the road-way beams. It is said that this alteration has added to the lightness and beauty of the structure.

CAST IRON SWING BRIDGE, PLYMOUTH.

Plates 92, 93, and 94.

The bridge is 91 feet in length from end to end of the parapet railing; its span is 47 feet, and the clear width of road-way between the guide plates is 8 feet 6 inches.

A section of the raised approaches is shown in Plate 92, and between

the ends of these approaches and the point from which the arch takes its spring, the stones are bedded and fitted to each other with great nicety, being connected with dowels, and otherwise prepared to receive the iron-work of the bridge.

Upon the foundations thus prepared, the bed plates are placed, the centre of each being in the centre line, or axis of the bridge, and 8 feet from the springing line.

An enlarged view of the bed plate is given in Plate 93; it is cast in one piece, true and even on the sides and edges, with eight flanges by which the plate is screwed down to the masonry, four being cast on the inside of the outer ring, and four at the centre of the plate, presenting the form of a cross. The upper surface of the ring is bevelled and turned true to fit the conical rollers hereafter described. The plate is let into the masonry to the depth of 3 inches, is laid perfectly horizontal, and screwed down with hold-down bolts and nuts, made of wrought iron in the Lewis form, with wedges.

The roller frame, as well as the other castings used in this bridge, is made of the best No. 2 pig iron, cast from the cupola, and of the toughest gray iron, free from air bubbles, or pin holes, and other defects. The eye of the roller frame is 6 inches in diameter, and truly bored to receive the pivot.

The holes in the rim are bored out to receive the roller axles or spindles; these holes are of an oval form, being 1½ inch in diameter in a horizontal or lateral direction, and 2 inches in diameter in a vertical direction. The outer rim is in twenty pieces, each of which is screwed to the arm flanges, with four screw pins of ½ inch diameter, so as to fix and keep the rollers in their places. See enlarged plan, Plate 93.

The rollers are in the form of the frustrum of a cone of which the base is 9 inches in diameter, and the altitude 4 feet 3 inches, the apex being in the axis of the pivot or centre of motion of the bridge. The altitude of the frustrum is 8 inches: the axles are of wrought iron; they are turned true, and fixed true in the rollers; that is, in each roller the axle is so fixed that their axes are coincident and their circumferences concentric.

The top frame, or upper traverse plate, is cast in two pieces, *i. e.* the arms and eye in one piece, and the ring and outer frame in another piece. The eye is 5½ inches in diameter, being truly bored out to receive the pivot, and the arms are neatly fitted and bolted to the ring and frame. The face of the upper traverse plate is also turned true in a similar manner

to that described for the bed plate, so that the bevelled surfaces of the ring and traverse plates may coincide, and lie fair with the bearing surfaces of the rollers, when all are placed in their respective positions.

There are holes in the frame, made in casting it, for the purpose of securing it and connecting it to the main ribs by screw bolts $1\frac{1}{2}$ inch in diameter, which are passed through the frame and through the ribs, and screwed up firmly by nuts.

The ribs forming the arched part of the bridge are four in number for each leaf of the bridge. They are placed about 2 feet 9 inches apart from centre to centre, and are each cast in one piece. A continuation of the ribs is carried across the upper frame to the circular tie-plate at the end, and bolted with nuts and screws to the flanges of the upper traverse plate, as shown in the transverse section, Plate 93.

In each leaf of the bridge are four cast iron tubes, placed transversely across the bridge, with collars abutting against the ribs; and through each line of the tubes passes a wrought iron tie, with a screw and nut at each end. These tubes and ties are shown on the plan, in Plate 94, and one of them is shown on an enlarged scale in the transverse section, Plate 93. A series of slight diagonal braces is also bolted by flanges from corner to corner of the compartments formed by these through ties passing across the bridge, as shown in the plan, Plate 94.

The flanges along the upper edge of the ribs are cast with holes to receive screw bolts $\frac{5}{8}$ inch in diameter, for fixing the road-way beams. The outside ribs have small rounded mouldings along the lower edge, and round the openings cast in it, as shown in the transverse section and in the elevation, Plates 93 and 94.

Between each pair of ribs, and at the end of each leaf of the bridge, next or over the abutments, for the purpose of connecting them together, there is a tie-plate, which is secured to each rib with flanges, bolts, and nuts; and at the junction, or meeting of the leaves, there is also a plate which is cast with flanges, by which it is secured to the ribs with nuts and screw bolts $1\frac{1}{4}$ inch in diameter.

The meeting or junction plate is cast in one piece, one-half straight and the other half curved, the straight half being a tangent to the curve, which is an arc of a circle whose radius is half the distance between the centres of motion, the right sine of the arc being half the breadth of the bridge. The curved end of one leaf is concave, and that of the other convex; the plates being cast with projecting wedge pieces and grooves, the wedge and groove on one being fitted and adjusted to the wedge

and groove on the other, so as to allow the two leaves to close evenly and firmly together.

The circular tie-plate for each leaf is cast in one piece; its form is the arc of a circle, the centre of which is in the centre of motion of the leaf.

An enlarged view of the skew back or abutment plate is given in Plate 93; it is cast in one piece, and is connected to the lower traverse plate by the radiating ties which are fitted into dovetail sockets in the traverse plate and let flush into the masonry, and the whole run solid with lead. A corresponding abutment plate, cast in one piece, is also fixed to the ribs, so that when the bridge is shut both plates fit firmly and closely together.

A circular road-way plate is let flush into the masonry at each end of the bridge, and secured by eight Lewis bolts and nuts to each plate, and run in with lead, the nuts and ends of the bolts being flush with the top of the plate.

On each side of the bridge is a railing, 3 feet 3 inches in height from the surface of the road-way.

The balusters are $1\frac{7}{8}$ inch square at bottom, diminishing to $\frac{7}{8}$ inch square at top; the top rail is of half-round or coach-tire iron, $2\frac{1}{2}$ inches broad by 1 inch thick.

The balusters are firmly riveted at top into the rail, and secured at bottom in their sockets by wedges and iron cement.

The road-way of the bridge, for its entire length and breadth, is covered with a planking of the best English oak, laid transversely, or at right angles to the line of the bridge, $2\frac{1}{2}$ inches thick, in one length, free from sap, wane, or other defects, well jointed, and screwed down to the flanges with twelve screw bolts, 5 inches long, with nuts under the flanges, two bolts through the plank at each flange. The bolts have countersunk heads, flush with the surface.

A sheet of patent hair felt, with a good coat of Archangel tar, is laid between the flange and the plank, and the whole screwed down firmly. Between the guide plates, a planking of sound elm, 2 inches in thickness, is laid over the oak planking.

Between the oak and elm planking also is laid a sheeting of patent hair felt, coated with Archangel tar, and the elm planks are well jointed and fastened down to the oak planking with 5-inch rose-headed spikes.

The guide plates are of cast iron, and extend along each side of the bridge.

Over the oak planks, and under the guide plates, is another sheeting of felt, coated with Archangel tar, and the plates are fastened down with screw bolts, $\frac{7}{8}$ inch in diameter, through the plank and flanges of the ribs, and the bolts screwed tight by nuts.

All the planking, previous to being used, was Kyanized, and after being fixed was payed over with two coats of Archangel tar and Spanish brown, well boiled together, and laid on hot.

The ballast, amounting to about 15 tons, is of cast iron, and intended to balance the bridge so that each leaf shall rest firmly on the rollers.

All the joints or openings in the castings were made up with iron cement, well caulked, and made solid throughout; the bridge being painted with three coats of lead-coloured paint and linseed oil.

GERRARD'S HOSTEL BRIDGE, CAMBRIDGE.

Plates 95, 96, and 97.

These Plates exhibit an elevation, plan, and details of this beautiful structure, which is probably one of the finest specimens of cast iron work, as applied to bridge building, which has yet been executed.

It was erected by the Butterley Iron Company, from the drawings of William Chadwell Mylne, Esq., F.R.S., son of the architect of Blackfriars' Bridge.

It crosses the River Cam, near King's College, in the town of Cambridge, and its appearance is in perfect keeping with the surrounding buildings.

In designing this bridge, the engineer seems to have taken the form of his arch from that of the celebrated marble bridge of the Santissima Trinitá at Florence, but the style of the structure is so admirably adapted to the situation that the design may be considered original.

The arch has a span of 60 feet, and is slightly pointed, if we may use the expression, since this is softened by the antique mask of a river god which ornaments its apex. It is formed by an ellipse from which a short segment is cut out at the flattest part of the curve, and the remaining portions are brought together by straight lines.

The water level coincides with the longest or conjugate diameter of the ellipse, from which level the arch springs.

The approach lies through an avenue of lofty trees, whose rich foliage, with the venerable buildings adjacent, and the quiet water of the Cam,

on which the bridge seems almost to repose, form altogether a picture of much beauty and interest.

We have already said that the span of the arch is 60 feet; its rise is 9 feet 6 inches; the length of parapet from end to end is 79 feet 3 inches, and the width of road-way between the parapets 8 feet 3 inches.

The construction of the bridge is as follows:

The foundations are laid upon piles 18 feet long by 9 inches square, shod with wrought iron shoes and straps.

Resting upon these piles are sleepers, which are placed at right angles to the line of the bridge, and fastened to the piles by ½-inch rag bolts.

Upon the top of these again are cross sleepers, placed in an opposite or longitudinal direction, and ranging in the same vertical planes as the rows of piles.

The planking rests upon these cross sleepers, and the foundations of the abutments upon the planking.

The earth was excavated from the under side of the cross sleepers to a depth of 2 feet for one abutment, and 5 feet for the other, and the space filled in with concrete, which extends 1 foot 6 inches beyond the line of masonry.

The abutments, which are built of stone, in the form of a horse-shoe, measure 10 feet 6 inches in length at the foundations.

The masonry at the end of the abutments, occupying the place of short wing walls, extends 10 feet from the face of the abutment. The abutments are carried up to the level of the springing with a vertical face on the inside, and a slight batter on the outside.

The springing line is 3 feet 6 inches above the top of the cross sleepers, and here the iron-work commences.

The abutment plates upon which the ribs rest are 10 feet 8 inches in length by 1 foot 9 inches wide, and 2 inches thick; a plan and section of the abutment plate are shown in Plate 96.

The road-way is supported upon three cast iron ribs, placed 4 feet 6 inches apart from centre to centre; each rib is cast in three pieces, which are connected by lapped joints, with eight screw bolts to each joint: these joints are shown in the elevations of the internal and external ribs in Plate 96, and a small figure drawn to a larger scale in the same Plate shows a plan of the over-lapping joint.

Plate 97 shows the flanges, bolts, and nuts by which the external ribs are secured to the abutment plates.

That part of each rib which rests upon the abutment is cast with a

triangular hollow; the dimensions of this hollow for the external and internal ribs respectively are shown in the elevations, Plate 96.

The centre piece of each rib is 2 feet 3 inches deep at its extremities, and 1 foot 6 inches at the centre; its section presents the form of a cross, the projecting pieces on each side serving to support the road-way plates: the external rib has only a projection on the inside, as shown by the small section in Plate 96, and by the enlarged transverse section in the same Plate.

Each external rib has eleven rectangular projections cast upon it, with four holes in each to receive the screws for securing the string course to the ribs: these projections are seen in the elevation of rib in Plate 96, and the mode by which the string course is attached is further seen by the enlarged transverse section.

Extending along the curved face of each external rib is a cast iron cornice 1 foot 2 inches deep, projecting 8 inches beyond the face of the rib, and bolted to it at top and bottom, as shown by the transverse section, Plate 96.

The internal rib, as seen by Plate 97, is provided with a small flange where it rests on the abutment plate; this rib is further secured and strengthened by two diagonal braces, one on each side; these braces are secured to the rib at top, and to the abutment plate at bottom, as seen in Plate 97.

The string course, which may also be termed the plinth of the parapet railing, is 79 feet 3 inches long by 10 inches deep, and projects $2\frac{1}{4}$ inches beyond the face of the rib; this plinth is secured to the external ribs by bolts with countersunk heads, in the manner before described.

The road-way plates, resting upon the projections of the ribs, as before described, have an area of 4 feet 4 inches by 4 feet 6 inches, and are $\frac{3}{4}$ inch in thickness; each plate is cast with three curved projections on its upper side; these projections extend in a longitudinal direction, and are 1 foot 6 inches apart from centre to centre; they are $\frac{3}{4}$ inch thick and 4 inches above the surface of the plate in the deepest part.

On two sides of each plate is a raised flange $2\frac{3}{4}$ inches deep, by means of which the plates are bolted to the ribs, as shown in the transverse section, Plates 96 and 97.

At the junction of the plates with each other a small under-lip projects, to form a bearing for the adjoining plate to rest upon.

The raised projections on the road-way plates divide the surface of each plate into four partitions; an elevation of one of the projections is shown

in the transverse section, Plate 96, and the small figure immediately above this transverse section is the section of a plate taken in a direction coinciding with the length of the bridge.

The road-metalling rests immediately on the plates, as shown in the cross section, Plate 97.

The railing of the bridge is cast of a rich lozenge-shaped pattern, with triangular-shaped openings, and is ornamented by a tablet in the centre, and corresponding pilasters at each end over the abutments.

Each length of the railing is composed of four pieces; namely, the plinth, the centre piece, the upper piece, and the coping. The plinth, with the mode in which it is fastened to the ribs, has been already described; the centre piece fits in between the open sides of the plinth, and is secured by the upper pair of screws already mentioned for connecting the plinth with the ribs; the upper side of the centre piece fits in the same way, between the open cheeks of the upper part of the railing, and is secured by bolts with countersunk heads: lastly, the coping cap fits over the upper part of the railing, being fastened with countersunk bolts 8 inches in length. These particulars, with respect to the mode of putting the railing together, will be fully understood on reference to the enlarged section in Plate 96.

FRIBOURG SUSPENSION BRIDGE.

Plates 98, 99, 100, and 100[a].

This bridge is erected over the valley of the Sarine to connect the hill on which stands the city of Fribourg with the opposite mountain. Before the construction of this bridge, the road leading through Fribourg to Berne and the German frontier of Switzerland descended into this valley, and gained the summit of the mountain opposite by an extremely crooked and precipitous route, on which were many inclinations exceeding 1 foot in 7. It will not appear strange that this road, at all times dangerous, was commonly quite impassable in winter. This state of things continued till 1830, when M. Chaley, a French engineer, undertook to erect a bridge across the valley for the sum of 300,000 francs, in addition to the profit of the tolls, which were conceded to him for the term of forty years.

It is remarkable that this bridge, exceeding in its length between the points of suspension that of any single-span bridge in the world, should be entirely constructed of fine wires, little more than $\frac{1}{10}$ of an inch in

diameter. By the combination of a material so delicate as this, it has been reserved for modern science to erect a bridge of the vast span of 870 feet between the suspension towers, which exceeds by more than 300 feet the opening of the famous Menai Bridge.

Fig. 1, Plate 98, is a general elevation of the bridge, showing that the chord of the curve assumed by the chain is 870 feet, and that the depth of the middle of the curve is 63 feet below the points of suspension.

MAIN SUSPENSION CABLES.

The wire employed for these is of the size called No. 18. The diameter of the wire is $\frac{12}{100}$ of an inch, and a lineal yard weighs 1·86 ounces avoirdupois. The platform of the bridge is suspended from four cables, namely, two on each side. Each of these cables is composed of 1056 threads of wire, and has a cylindrical section of 5½ inches in diameter. The two cables on the same side of the bridge are only separated by an interval of 1½ inch, which is the space occupied by the heads of the suspension cords. The length of each suspension cable is 1228 feet, and at every two feet of its length it is firmly bound by a ligature of wire, which preserves the cable in a cylindrical form. These ligatures of wire are continued for those parts of the cable which are behind the piers of suspension, as well as for those between the piers. In approaching these piers, the two cables on each side of the bridge gradually spread out and unite into one flat band of parallel wires, which in this form passes over the three friction rollers on the top of the pier. These rollers, and their position on the pier, are seen in plan and elevation in figs. 2, 3, and 6, Plate 99; and figs. 1 and 2, in Plate 100, are elevations of the rollers on a larger scale. After passing over the three rollers in the form of a flat band entirely covering the breadth of the rollers, each main cable again separates into four smaller cables, which are each composed of ten bundles or strands of wire, and near the surface of the ground these are attached to the mooring cables. These mooring cables are sixteen in number, that is, eight for each end of the bridge.

Fig. 4, in Plate 100, shows the two mooring shafts at each end of the bridge. In each of these mooring shafts are four cables, which pass through a strong anchor of masonry, which will presently be described. Each mooring cable is 4 inches in diameter, and is composed of 528 wires, so that the whole mass of wire in the four cables on each side of the bridge is the same as in the two suspending cables with which they are united. The whole of that part of the cables which passes through the stone-work of the shafts is enclosed by a coil of single wire, wound closely

round it in a spiral form. The main cables, after having passed over the heads of the piers, are called retaining cables. These retaining cables are four in number on each side of the pier, and each of these, as before described, consists of ten separate bundles. Figures 10 and 11, in Plate 100, show the method by which ten of these bundles are attached to one of the mooring cables. Each of these ten bundles passes over a stirrup iron or crupper of a semi-cylindrical form, and is bent back in the form of a loop, and firmly bound by a larger size of wire. The mooring cable is bent round a similar crupper of larger dimensions, and then, the whole of the eleven cruppers at each junction being adjusted so that the opening through them is quite clear and uninterrupted, this opening is occupied by three wrought iron keys, two of which have heads and the middle one is plain, being driven in as a wedge to fix and tighten the other two. Figs. 12 and 13 show the form and dimensions of the crupper for the mooring cables, and fig. 11 shows the position of the several cruppers and of the keys which secure them to each other. After being united to the retaining cables, the mooring cables descend without changing their direction into the interior of the sloping galleries, and when arrived at the entrance of the vertical shafts, each of them spreads out into a flattened form, and passes over a friction roller, so as to drop vertically into the orifices in the mooring shafts. Figs. 5 and 6, Plate 100, show the form and position of one of these rollers at the entrance of the mooring shaft. For the support of these rollers, two blocks of granite, 7 feet 8 inches in length by 3 feet 3 inches in breadth, are bedded in a sloping position on the natural rock. Each of these blocks carries four cast iron plates, upon which are fixed the four friction rollers for each side of the bridge. These rollers are 16 inches in diameter and the same in length.

SLOPING GALLERIES.

On the side of the town, considerable difficulty was experienced in driving these galleries, as they had to pass through a mass of loose and crumbling rock, on which stood many old and dilapidated houses. The excavation of these galleries, however, was successfully performed, and at both ends of the bridge they were roofed with arches of limestone. Each of the four galleries is 6 feet 6 inches in width and the same in height. The space between the entrance to these galleries and the main piers of suspension is levelled off to a flat surface, and forms a terrace, which extends about 7 feet in front of the main piers, and is supported by retaining walls of limestone about 17 feet in height. The front of

this terrace, overlooking the valley, is in the form of a demi-lunette, as seen in the general plan, Plate 98. The surface of the terrace is on a level with the platform of the bridge; and on approaching the small parapet which surrounds it, the whole expanse of the valley is open to the spectator, and presents a combination of natural and artificial beauties which together make up a picture certainly not inferior to any in the world.

MOORING SHAFTS.

These are sunk entirely through the solid rock. They are four in number, 52½ feet in depth, and 10 feet in length by 3 feet 3 inches in breadth. Fig. 5, in Plate 100, is a transverse section of one of these shafts, showing the arrangement of the stone blocks into three separate inverted arches, with straight shafts between them. Excavations are made in the sides of the shaft to receive these arches, and the admirable resistance opposed by this species of stone anchor will be readily understood on reference to Plate 100. A vertical opening is left through each of the middle stones for the passage of the mooring cables, and at the bottom of the shaft these are firmly secured by keys and stirrup irons, as shown in figs. 8 and 9. Each of these mooring shafts was formed with a service or working shaft, about 3 feet square, which was left open after the masonry was finished. The use of this working shaft in fixing the mooring cables will be seen hereafter. The masonry in the mooring shafts is composed of Jura limestone of the best quality. In order to communicate with the base of these shafts, a gallery of communication at each end of the bridge was driven from the side of the valley, for the length of about 350 feet. These small galleries are about 8 feet in height and 3 feet wide. They are driven in a direction coinciding with the centre line of the bridge, and when arrived at the line joining the mooring shafts, a short cross gallery is driven right and left to the base of each shaft.

MAIN PIERS.

The main pier of suspension at each end of the bridge is in the form of a Doric portico, which serves as an entrance to the road-way of the bridge. Each front and side of the portico is ornamented with an entablature and pilasters, as shown by fig. 1, Plate 99. The opening is surmounted by a semicircular arch, and is 43 feet in height by 19 feet in breadth. The masonry of this portico is cramped at every course, and is founded upon the rock, 16 feet below the surface of the ground. The basement of the two piers of each portico is 26 feet in height from the foundations. The

courses of the basement are faced with Jura limestone, and the hearting is composed of solid blocks of sandstone, dressed to perfectly regular figures, so as to leave no hollows at the joints and angles. The part above the basement is built entirely of sandstone. Figs. 1, 2, 3, 5, and 6, in Plate 99, exhibit the form and dimensions of the portico, and fig. 1, Plate 100, shows the granite blocks supporting the friction rollers, which have been already described. In addition to the cramping of the courses in the main piers, numerous keyed ties, with heads in the form of a cross, are let into the masonry to bind the whole together in the most substantial manner.—See figs. 10 and 11, Plate 100[a].

VERTICAL SUSPENSION CORDS.

Each of these is composed of thirty threads of wire, and presents a diameter of 1 inch. Their length varies according to their position, the shortest being about 6 inches and the longest about 54 feet. These suspension cords, to the number of 163 on each side of the bridge, are placed at equal distances of 4 feet 11 inches apart. Each cord is terminated at both extremities by an annular crupper, over which the cord is bent. The crupper at the lower extremity receives the hook of the strap which passes under the road-way beam, and the crupper at the upper end is supported upon the middle of a double saddle, which rests upon the two main cables over each of the vertical cords. Figs. 5 and 6, in Plate 100[a], show on a large scale the attachment of these suspending cords to the beams of the road-way and to the main cables.

ROAD-WAY.

The breadth of road-way between the railing on each side is 21 feet 3 inches, of which 15 feet 5 inches in the middle are occupied by the carriage-way, and the remaining breadth on each side by a narrow foot-path. The road-way beams are 9 inches in breadth, 14½ inches in depth in the middle, and 12 inches at the sides under the foot-paths. This diminution of thickness gives a slight curvature to the top of the beams, and this curvature, which is preserved in the surface of the road-way, is sufficient to allow the water to run off to the sides. The road-way beams are placed 4 feet 11 inches apart from centre to centre. The longitudinal planking for the carriage-way rests upon the road-way beams. The planks composing it are 3½ inches in thickness by 6 inches in breadth, and from 30 feet to 60 feet in length. A small space is left between the sides of each plank, as shown in the detailed section in Plate 99. This longitudinal planking is covered by transverse planks 2 inches in thickness, and these form the surface of the road-way. The foot-paths are raised about

7 inches above the carriage-way; they consist of close transverse planking laid upon longitudinal joists which rest upon the transverse road-way beams. These joists are of unequal depth, so as to give the foot-path a slight inclination towards the carriage-way. The timbers of the railing on each side of the bridge are fixed into the outside joists of the foot-paths. The rails are placed in a sloping direction, without upright posts, forming a series of St. Andrew's crosses, and the tops of these are mortised into a hand-rail, rounded off at the top, as shown in the detailed section. Every fourth road-way beam projects about $3\frac{1}{2}$ feet beyond the others, in order to afford a bearing for the diagonal iron ties to support the railing in a vertical position. These iron ties are bolted to the ends of the beams and to the hand-rail, as shown in the detailed section. The platform of the bridge has a slight curvature across the valley, being inverted to the curve of the chains, so that the rise in the middle part varies, according to the temperature, from 20 inches to twice that height above the horizontal line joining the two extremities of the platform.

FABRICATION OF THE CABLES.

Preparation of the wires.—The coils of wire, as delivered on the works, weighed generally 18 or 20 ℔s., and the whole length of each was minutely examined on its arrival. If found to be without defect, it was immersed during two hours, three several times, in a cauldron of boiling linseed oil mixed with a small quantity of litharge and soot. On coming out of the cauldron, the wire was hung on lines to dry, and when the drying was complete, it was returned to the depôt to await a new operation. Every thread being thus covered with three coats of oil is quite inaccessible to oxidation, so long as it is not exposed to friction.

Winding and joining the threads.—The coils of thread, after being thus varnished and dried, were rolled upon drums or reels about 16 inches in diameter. In performing this operation, a workman fastened one end of a thread to the reel, and turning it round by means of a winch with his right hand, directed the wire with his left. On coming to the end of a length, he united the ends of the two wires by twisting them together for about 4 inches, and binding this twisted part tightly round with annealed wire, size No. 4, disposed in a spiral form with continuous folds. These junctions were so securely made, that, on subjecting any length of wire to an experimental strain, the ends never slipped from each other, and the wire always broke at some place between the joinings. An expert workman could join and wind per day about 1100 ℔s. weight of wire, which quantity would fill three of the reels.

Main suspension cables.—The grand walk in which the strands for the suspension cables were fabricated not being of sufficient length in a straight line, these great cables, 1228 feet long, were made in double lengths. It will be remembered that the suspending cables are four in number. Each of these is composed of twenty strands, namely, twelve of fifty-six threads each, and eight of forty-eight threads. Each strand was separately formed in the following manner.—See fig. 7, Plate 100*.

A rectangular block of oak (b) is fixed upon a low frame and retained in its place by a strong counter-arch abutting against the direction in which the wires are to be strained. To the outside of this block, at about 3 feet in height, is bolted a semi-cylindrical piece of wood 14 inches in diameter and cased with sheet iron. At the distance of 614 feet from this block are fixed, about 3 feet apart, two other blocks of oak (a) and (c), from each of which projects a hook to receive the cruppers which terminate each of the strands. Throughout the distance of 614 feet transverse cylinders are placed about 30 feet apart, to support the wires of the cables.

Every thing being thus prepared, the cruppers are hung upon the hooks, and one extremity of the wire is attached to one of the blocks, as, for instance, to (a), and being passed through one of the cruppers, is carried off in a cart bearing the whole reel with the wire wound round it. Arrived at the other end of the walk, the thread thus unwound is bent round the semi-cylinder at the outside of block (b), and after giving to this thread a tension of 220 ℔s., the cart is conducted back to the starting point; there the thread is passed round the other crupper, and subjected to the same tension as the other length: this operation is continued in the same manner to the end of the strand. The end of the thread which completes the strand is then to be united to the first thread which had been temporarily fastened to the block (a) in commencing the operation. In order to obtain the proper tension for each thread of the wire at each end of the walk, a pair of pincers is attached to the wire and to a cord passing over a horizontal cylinder at each end of the walk, and having a weight of 220 ℔s. attached to it. These are the cylinders marked (c) and (d) in fig. 7. Every part of the wire composing the strand is in this way subjected to the same tension.

The two parts of the strand close to the cruppers are then bound for a length of 18 inches by a spiral ligature of annealed wire, No. 14. In addition to this, temporary ligatures are wound round the strand at about every 3 or 4 feet apart, and these are not removed till the strands are about to be united into one great cable. In this state the whole of the

strand thus bound together is payed over with a coat of the same oil varnish through which the wire had already passed three times, and the strand is then laid at full length along the walk by the side of the others already made.

Five workmen, of whom one is required to apply the tension, can fabricate in this manner five strands per week.

Mooring cables.—These were made under shelter in a covered walk about 85 feet in length. In this walk was dug a longitudinal ditch, 20 inches in breadth and 3 feet 4 inches in depth. In this ditch was placed a line of beams 82 feet in length and 12 inches square; 16 inches above this line of beams was placed another line of similar dimensions, supported by uprights at about every 10 feet, and strongly strapped to these uprights.

The extremities of each of the beams abutted against uprights of oak, the tops of which stood a few inches higher than the surface of the upper beam. In the head of each of these uprights is bored a hole 2½ inches diameter, the lower part of which is exactly on a level with the surface of the beam. In this hole works the screw bolt *a a*, 2½ inches diameter and 31 inches in length (see figs. 1 and 2, Plate 100[a]). This bolt carries a screw for about half its length, and the other end is attached to the crupper of the cable. By means of a small lever about 3 feet in length, a nut, through which the screw passes, is worked round so as to adjust the end of the bolt *a a* to any required position during the fabrication of the cable. The small crane *b* (fig. 2, Plate 100[a]) is employed to move the weight which each wire was required to support at the instant of its inflexion upon the crupper. The vertical post of this crane swings round as a pivot, and the end of the horizontal arm carries a small pulley.

In commencing the manufacture of a cable, the extremity of one of the wires wound upon a reel is attached to one of the beams and passed through the neck of the crupper. The wire is then carried to the other extremity of the walk, and supported against the neck of the other crupper; about 2 feet beyond this the wire is clasped by a pair of pincers attached to a cord which passes over the pulley on the horizontal arm of the crane. To the other end of this cord is attached a shell weighing 220 lbs., which ordinarily rests upon the ground. When the strain is to be applied to the wire the crane is swung round on its pivot, so as to tighten successively the cord and then the wire: it is evident that at the moment when the arm of the crane stands out at such an angle as to force the whole length of wire and cord into a perfectly

taut condition, the weight will be raised from the ground and supported by the wire and cord. By the same process, and with a similar apparatus, the weight is raised by the wire at the other end of the walk. Whenever the least variation is found in the distance between the two cruppers, whether arising from alternations of temperature or other causes, this distance can be exactly re-adjusted by means of the screws, already described, at each end of the walk. When all the threads of the cable have been thus stretched and placed together, the end of the wire is united to the first end which had been temporarily attached to the beam. In this state each cable was well payed over with a coating of oil varnish, prepared as before, and this varnish was forced as much as possible into all the vacuities between the wires.

The cable was then strongly bound with annealed wire, No. 14, at the extremities of the cruppers. This wire ligature was continued for about 2 feet in length. The threads of the cable were then bound into one solid bundle by a close spiral envelope of wire for the whole of that length which was destined to be placed in the mooring shafts. The remainder of the cable was bound at every 2 feet with a ligature for the length of 8 inches, and in order to press these ligatures more tightly, and to render the cable more cylindrical, a kind of circular vice was contrived to fit the figure of the cable, and the two parts of this vice being forced together by means of a screw, a great pressure was applied to every ligature.

When the first cable had been thus fabricated, it presented a most satisfactory appearance; but it was no sooner taken from the frame on which it had been stretched than the elasticity of the great mass of wire which composed it came into play, and caused it to assume a series of double curves like those of a corkscrew. It would have been impossible to force it in this state into the opening of the mooring shaft. After many ineffectual efforts to maintain the cable in a straight line when at perfect liberty, in order that it might be placed in its position in the mooring shaft, M. Chaley adopted the following expedient. He caused to be prepared a number of small deal laths cut from green wood; these laths were $\frac{3}{4}$ of an inch thick and 2 inches wide, and the whole length of the cable being enveloped in four thicknesses of these, the whole mass was firmly tied round with ligatures of annealed wire at every 9 or 10 inches. Thus enveloped in a case of wood, the cable was left to itself, and experienced no change except a very small amount of torsion. This kind of packing round the cables had the further

advantage of protecting them from injury during the operation of fixing them in their places.

Eight cables of the same length, intended for the two sides of the valley, were successively fabricated and packed round in this manner. Fig. 1 is a section of one end of the frame, showing the long beams, the upright, the tightening screw, and crupper. Figs. 2 and 3 are an elevation and plan of one end of the frame, showing the long beams, the upright, the crupper, and part of the cable in course of manufacture; also the crane, with the weight suspended over its pulley. The crane is here shown in the position it takes when the weight is being supported by the wire. The dotted lines on the plan and section show the position of the arm of the crane when the weight is at rest on the ground.

Suspension cords.—After having calculated the different lengths of these cords, and projected their lengths upon a scale of $\frac{1}{5}$ the full size, M. Chaley proceeded to manufacture these in sets of four for each different length required. For this purpose two posts were firmly fixed in the ground at a distance of 55 feet apart, and throughout the whole length between them a horizontal bar was fixed and supported at intervals by uprights. An iron hook was fixed vertically into one of the posts, and held the small fixed crupper of the system. A small waggon which ran upon the bar contained a corresponding moveable crupper, and could be fixed at pleasure to the bar or allowed to slide along it. In the fabrication of the cords the waggon was fixed successively at the several required distances from the fixed crupper, and the wire, to the number of thirty threads, was continually doubled round them, and the proper tension applied to each single thread by means of a screw. The threads were then tied together at the cruppers, and the whole length of the cord bound by a spiral thread with folds about an inch apart.

METHOD OF RAISING AND FIXING THE CABLES.

When all the masonry of the main piers and the mooring shafts had been completed, the sloping galleries opened, and all the friction rollers fixed in their places, the work of fixing the suspension cables was proceeded with.

For this purpose a scaffold was established upon each of the main piers, projecting on each side of a hollow which had been left above the arch of the portico. Upon this scaffold were fixed two windlasses, one before the other, the barrels of which were 10 feet long and 10 inches diameter. The position of these windlasses is seen in figs. 3 and 6,

Plate 99, and in fig. 4, Plate 100ª. In the latter Plate the two windlasses, for the convenience of reference, are numbered 1 and 2. Six levers or capstan bars, 5 feet long, were employed at each extremity of the barrel for working these windlasses. Another windlass (No. 3, Plate 98), of the same size as the first two, was firmly planted in the axis of the bridge, about 33 feet beyond each of the main piers. A hempen cable, about 760 feet in length, and something more than an inch in diameter, was wound upon the axle of No. 3 windlass. The free end of this cable was then conducted to the main pier, and after two revolutions round the axles of No. 1 and No. 2, it was attached to the end of a smaller rope, the other extremity of which was fixed to the windlass No. 4, placed in the bottom of the valley in the axis of the bridge, and nearly under the middle of its length. About 170 feet of the smaller rope were then wound on the axle of No. 4. The same manœuvre was executed for the other side of the valley by a similar apparatus of ropes and windlasses. The windlass for this other side is marked No. 5 in Plate 98.

The first strand of the suspension cables was then brought between the two windlasses, No. 4 and No. 5, which were placed, as we have said, in the bottom of the valley. For this purpose a large cylinder or drum (see figs. 8 and 9, Plate 100ª), 6½ feet in diameter and 5 feet long, was mounted upon a low car with four wheels, and so placed as to turn freely on an iron axle. This car was conducted to the walk where the strands had been made, and where they were laid out in double lengths. One strand taken up by the middle was then attached to the drum, and a motion being given to the car, the whole strand was rolled upon the drum in the form of a doubled rope. The strand was then conducted to the spot from which it was to be raised to the main piers. Arrived at this spot, the car was firmly fixed between the two windlasses No. 4 and No. 5, as shown in the plan, fig. 9, Plate 100ª. The two cruppers or extremities of the strand were then strongly secured, one on each side of the car, to the hempen cables, near the point where these cables were rolled upon the axles of No. 4 and No. 5. The whole of the windlasses at each end of the bridge were then set to work, and gradually the great hempen rope was wound up, carrying with it to each end of the bridge one extremity of the suspension cable, and causing it at the same time slowly to unroll off the drum. When the whole of the strand had been thus unrolled it quitted the drum, and the car was then at liberty to be drawn back to the walk to take up another strand.

When the two extremities of the strand had thus arrived at the main piers, the workmen ceased to heave at the windlasses on one side of the valley, but continued working at those on the opposite side. The strand was thus drawn over moveable wooden rollers placed upon the main pier, and was there attached near its crupper to another hempen cable, which was wound upon the axle of windlass No. 6, placed at the bottom of the sloping gallery. From this position the strand was removed without much effort on to one of the friction rollers at the top of the pier. The workmen then hove away at the windlasses on the other side of the valley, and the other end of the strand was drawn into the sloping gallery on that side by means of windlass No. 7.

Two bench marks had been fixed precisely at the same level, one on each of the main piers, to indicate the flexure which should be given to the suspension cables. The heaving at windlass No. 7 was continued till the line of sight from one bench mark to the other became a true tangent to the inferior part of the catenary assumed by the strand.

Each mooring cable, packed round with laths as above described, was then carried by thirty men to the entrance of the mooring shaft, and placed in the sloping gallery upon wooden rollers placed there to receive it. A strong hempen rope, wound upon the axle of the windlass placed near the entrance of the mooring shaft, was then led down the service or working shaft of 3 feet square, bent over a pulley placed at the bottom, and brought up through the small opening destined to receive the first mooring cable. The free end of the rope was then firmly bound with wire to the crupper of the mooring cable, and the windlass being set to work, the hempen rope, on its return, drew with it the mooring cable through the opening left in the masonry. Great care was necessary in this operation; and in order to prevent curves or torsion of the cable, it was performed very gradually, the wooden envelope of the cable being regularly cut away as it descended into the mooring shaft.

In the interior of the sloping galleries cross pieces had, in the mean time, been placed out of reach of the bars of the windlasses, to support the first suspending strand which had already been fixed; and the windlass No. 6 was thus free to be employed in raising another strand of the main cable. A second strand being then raised by a process similar to that which has been described for the first, the two cruppers of these strands were placed one on each side of the mooring cable, and secured to it by keys, as before explained, and shown by figs. 10 and 11, Plate 100.

The whole of the forty strands forming the two cables for one side of the bridge were first fixed, and then those for the other side were raised and fixed by the same means. When all the strands of the suspension cables had been thus fixed, they formed two bands, each 31 inches in breadth, with a space of 30½ feet between them. The cables continued in the form of a band from their junction with the mooring cables to the place for attaching the first or longest of the suspension cords. Then throughout that part of their length from which these cords were suspended, the forty strands were divided into two equal masses of twenty each, and held by means of a screw vice in this position whilst they were bound by the wire ligatures intended to preserve them in the cylindrical form. Each suspension cord placed between the two cables, and supported by the saddle or cushion resting on the cables, carried at its lower extremity a stirrup iron to receive the transverse beams of the road-way.

After the platform of the bridge was fixed, the cables were payed over with a varnish prepared as before described, which was made to penetrate as much as possible, and to cover them all over. The suspension saddles, stirrup irons, and cords, as well as the retaining and mooring cables, were payed over all their exposed surfaces. When the ligatures were every where completed and the whole bridge finished, all the cables were finally painted with white lead and oil. The white colour weakens the action of the sun, and allows any symptom of incipient oxidation readily to manifest itself. Abundance of oil has been regularly supplied to that part of the main cables which passes over the friction rollers on the main piers, and this practice has been strictly attended to from the first opening of the bridge to the present time; and in order to prevent oxidation of the mooring cables, liquid grease is periodically poured into the openings in which these cables are fixed.

MATERIALS USED IN THE CONSTRUCTION OF THE BRIDGE.

With very slight exceptions, Switzerland itself has furnished all the materials and all the workmen employed in this bridge.

The Jura limestone for the masonry of the main piers and the mooring shafts was procured from the quarries of Neuville and Lengnau, in the chain of the Jura mountains, distant 10 or 15 leagues from Fribourg. The stone was transported by land carriage to the banks of the lakes of Neuchâtel or Bienne, and conveyed by water along the line of the River de la Broie and the Lake of Morat to the town of this name, which

is only three leagues from Fribourg. For this last part of the distance the materials were conveyed by land to each end of the bridge.

The sandstone of which the mountains on each side of the valley are entirely composed was found an excellent material for building. Two-thirds of the whole masonry of the bridge consists of sandstone procured from quarries less than a mile west of the town. From these quarries blocks of any size could be obtained without any limit, except the difficulty and expense of transport. This stone, when first quarried, was so moist and soft that the masons could dress it with great ease. After being exposed to the air, however, for several months, it dried to a bluish-gray colour, and acquired a considerable solidity and hardness.

The limestone of which the retaining walls in front of the terraces were built is a calcareous tufa, composed of very hard stalactites whose cells are sometimes empty and sometimes filled with a calcareous matter, which is almost of a pasty consistence in the quarry, but which very quickly hardens in the air, and acquires great firmness even in subterranean works.

This stone was procured from the quarries of Corpateau, on the banks of the Sarine, 6 miles south-west of Fribourg.

The granite used for the support of the friction rollers was procured from isolated masses scattered over the surrounding country. Its quality is very variable.

Before commencing the works, specimens of the different kinds of stone were submitted to experiment with the hydraulic press.

The following is the result of these experiments, with the weight of a cubic foot of each.

	Weight borne without injury per square inch.	Weight in lbs. of a cubic foot.
The Jura limestone	3307	188
The sand or gritstone	555	137
The calcareous tufa, about	555	118

The iron was principally procured from the works of M. Finot d'Undervillers, in the canton of Berne. The wrought iron ties, however, for the masonry of the piers, and the screw bolts for securing the timbers of the platform, were supplied by the rolling mills of England. Notwithstanding the expensive carriage of these articles by sea and land, their cost was at least 20 per cent. under that of French iron, although the latter is extensively manufactured in Franche-Comté, within thirty leagues of Fribourg.

The whole of the iron wire was supplied from the wire works of Messrs. Neuhaus and Panserot, of Bienne, who manufacture all their wire from the iron of Undervillers.

The fir timber of which the road-way is constructed was furnished in abundance by forests in the canton of Fribourg, and sawn at a neighbouring mill worked by a fall of water.

Such is the celebrated bridge of Fribourg, a work which we have thought worthy of a somewhat lengthened description, not less because the bridge is in itself one of the finest examples of engineering skill which the world has ever seen, than because it is the principal model which this treatise presents of a very important division of bridge architecture. For most of the particulars of this description we are indebted to an able notice in the Annales des Ponts et Chaussées, by M. Chaley, the engineer already mentioned as the constructor of the bridge.

THEORY OF THE ARCH.

Plates 101, 102, and 103.

(See Professor Moseley's Theoretical and Practical Papers, vol. i.)

CHAIN BRIDGE UPON THE CATENARIAN PRINCIPLE.

Plates 104, 105, 106, and 107.

This is a design by Robert Stevenson, Esq., of Edinburgh, exhibiting a novel application of the catenarian curve to the construction of chain bridges. This design, it will be observed, has never been carried into execution, but was proposed by Mr. Stevenson for the great north road between Edinburgh and Queensferry, where it crosses the river Almond. The chief peculiarity which distinguishes this from the chain bridge of suspension is the mode in which the chains are fixed without passing over high turrets, as in the ordinary chain bridge. Mr. Stevenson's bridge may be termed a suspension bridge with reference to the catenarian curve or curve of suspension assumed by the chains; but with respect to the road-way the term suspension can no longer be applied, because the road-waw is raised *above* the chains, and rests *upon* them, instead of being suspended from them.

The main chains in this design are five in number, and each is made to collapse or turn round the abutments of masonry, as shown in Plate 106. The chains are also continued through horizontal passages in the abutment, quite up to the face of the latter, where the end of each chain is formed into a great nail or bolt. The countersunk or conical heads of these bolts are made to fit into corresponding hollow tubes of cast iron, fixed in the masonry of the abutment. The superstructure resting upon the catenarian chains consists of a series of iron arches, with iron columns between. The span of these arches varies from 3 feet at the sides to 5½ feet in the middle of the curve.

The plan and section in Plate 105 show the transverse bearers resting on the main chains, with the diagonal bracing between the columns. It will be seen from this plan and section that the bearer in the middle of the curve is 42 feet in length, while the length of those at each end is no more than 32 feet, that is, the width across the five chains. The additional width of 5 feet on each side, which is given to the middle bearers, is suspended from the level of the road-way by diagonal bars, as shown in the transverse section, Plate 105. The intermediate bearers on each side of the middle one project a proportionate distance beyond the outer chains, commencing at 5 feet in the middle and diminishing to nothing at the ends. The tapering form thus given to the bridge, both in plan and section, would increase its lateral stiffness, and serve in some degree to prevent that vibration which is so injurious to light chain bridges. Plate 107 contains a section and plan showing the galleries formed in the abutment for the admission of the main chains. These galleries are 5 feet wide and 7 feet high, arched with semicircular tops, and each one can be entered by means of the arched access, or gallery of access, which passes at the back of the abutment and communicates with all the longitudinal galleries containing the chains.

The design exhibited in these Plates is applicable to a bridge of 150 feet span, and Mr. Stevenson observes, in reference to bridges of this kind, that the expense of making[4] up the road-way, and the enlarged angle of its suspension, may be considered as limiting the span or extent of bridges of this construction to about 200 feet. The justice of this will be evident when we consider that the height from the curve to the road-way must necessarily be increased according as the span is made greater; so that for bridges of very large span the cost of the superstructure would render

[4] Description of Bridges of Suspension. By Robert Stevenson, Esq., F.R.S.E., in the Edinburgh Philos. Journal, vol. v. p. 237.

these bridges more expensive than chain bridges with turrets for suspending the chains. At the same time it is evident that, for any span less than about 200 feet, Mr. Stevenson's principle possesses many important advantages, amongst which may be mentioned the simplicity of the construction, and the facility with which any particular chain may be withdrawn and replaced without injuring the fabric of the bridge.

DARLASTON BRIDGE, STAFFORDSHIRE.

Plates 108, 109, and 110.

This is a bridge of one arch, with a span of 86 feet and a rise of 13 feet 6 inches. The length of the parapet from end to end is 117 feet, and its height 4 feet. The road-way is embanked at each end of the bridge to the length of about 15 feet, and measures 26 feet 6 inches in width between the parapets.

The bearing piles are of 4-inch planking, 8 feet in length, and are placed in rows 4 feet apart from centre to centre, both in a transverse and longitudinal direction under the abutments and wing walls. Upon each internal row of piles a sill with a scantling of 12 inches square is bolted to each pile. In front of these sills are driven sheeting piles 11 feet long by 4 inches thick; these sheeting piles are spiked to the sills, and are further fastened and supported by waling pieces, 12 inches thick by 9 inches wide, passing round the outside and secured to them by screw bolts fixed at intervals of 4 feet, passing through the sills as well as the sheeting piles, and made fast by nuts.

Sleepers 12 inches wide and 6 inches thick are spiked to the bearing piles in a transverse direction, and are crossed by others of similar scantling extending in a longitudinal direction, and spiked to the under ones, each course of sleepers being made to butt against and fit close to the sills before described. The sleepers are covered by the platform or flooring, which consists of 4-inch close-jointed planks, spiked to the upper course of sleepers, and extending to the outside of each sill.

The abutments are founded on the platform at 11 feet 6 inches below the water level, and are raised upon three courses of footings, the lowest of which is laid horizontal, and is 15 inches in thickness; the other courses are laid with horizontal quoins and faces, but radiating internally in a transverse direction to a point at the level of the upper side of the lower course, and 11 feet 6 inches from the general face of the abutment.

The footings project 6 inches on each face, the lowest one measuring 34 feet in length at the under side.

The stones forming the face of the abutments are in level courses 1 foot 6 inches in thickness at the footings, and decreasing in thickness to the springing course, where they are 12 inches deep.

The interior of each abutment is built solid, with slanting courses whose beds radiate to the same point as described for the footings.

The abutments are strengthened by counterforts, one built in a line with each end, and a third one in the centre line of the abutment; they are founded at the same depth as the abutments upon three courses of footings, also similar to those of the face of the abutments, and are carried up solid in horizontal courses, diminishing in thickness from 18 inches to 1 foot, and finished off at the top with a slight longitudinal inclination. These counterforts are connected to each other by semicircular counter arches, which are embedded into the masonry of the abutment. The springing course of each abutment is 12 inches in depth, and projects 3 inches in front of the face of the arch, being washed off to the depth of 3 inches at the top.

Each outside front of the abutments is embellished by four Tuscan columns, placed in couples, with a niche 9 feet high and 4 feet wide in the centre; these columns rest upon the level of the springing course, and are surmounted by a frieze 15 inches in depth, which extends the whole length of the bridge.

The wing walls are built in courses varying from 15 to 18 inches in thickness; they are curved outwards on the plan and made to sweep downwards in elevation, being covered by a chisel-dressed ashlar coping, 3 feet wide and 6 inches deep. They are terminated at each extremity by a square pilaster, measuring 3 feet at the top and 2 feet above the surface of the ground, surmounted by a pointed coping cap of ashlar, 2 feet deep, and projecting 3 inches each way.

The stones forming the arch measure 18 inches on the face; they are worked into the abutment at the springing, and are 3 feet 6 inches in bed at the crown of the arch. The extrados of the arch is worked off to a tangent at the haunches.

The face courses of the arch are of ashlar, worked to the radius of the arch, with their ends cut to suit the vertical joints in the horizontal courses of the spandril walls. The whole of the front ashlar of the arch, the spandrils, the pilasters, and wing walls, have all the outside joints chamfered, as seen in Plate 108.

The course is 18 inches deep, 3 feet 4 inches in breadth, with a projection of 18 inches. The plinth of the parapet projects 3 inches each way, and is 15 inches high. The dado is 14 inches thick, with a coping 10 inches deep, and washed off at the top. Each end of the parapet is terminated by a pilaster, with a tablet in the centre of a corresponding character with the columns and niche beneath, and a similar pilaster and tablet are placed over the crown of the arch.

Plate 110 comprises an elevation and section of the centering used in the construction of this bridge; it consists of five ribs, placed 6 feet 6 inches apart from centre to centre; each of these ribs is sustained by piles driven 10 feet 6 inches apart in a longitudinal direction.

These piles are connected by sleepers 9 inches in thickness, 15 inches wide, and 28 feet 6 inches in length, extending the length of the arch, and spiked to each pile. Small upright bearers, 2 feet 3 inches in length, are fixed firmly upon the sleepers, over each pile; these are surmounted by eight connecting beams, similar in dimensions, and corresponding to the sleepers just described.

The wedges are of the most simple form, as shown in the section, Plate 110; they are placed upon the connecting beams, two over each pile or bearer, thus forming the support for each separate rib, which is constructed in the following manner:—the straining beam, extending from one springing to the other, takes its bearings upon the wedges, and is strengthened in the middle by an iron strap on each side, 4 feet long by 3 inches wide, connected by four bolts, passing through each strap, as well as the beam.

The queen-posts which support the filling in pieces are placed over each bearer, and secured upon the straining beam by strap irons; these are kept in their places by diagonal braces, extending from one queen-post to the other, and from the bottom of the queen-posts to the filling in pieces, with bolts driven through the intersections, and secured by strap irons and bolts, as shown in the elevation. The filling in pieces are also secured to the straining beam at each springing by strap irons, with four bolts through each.

BASCULE BRIDGE, WELLESLEY LOCK WORKS.

Plates 111 and 112.

This is a draw-bridge, consisting of two leaves which meet in the middle, and are raised by means of crab machinery placed at each end of the bridge. The span of the arch is 40 feet, its rise 2 feet 3 inches.

The bridge contains eight main ribs, each of one casting, 2 inches in thickness. Elevations of one of the internal and one of the outside ribs are seen in Plate 111. The latter are ornamented by scroll-work, as seen in the Plate. The spaces between the two outside ribs and those next to them measure 4 feet 4 inches in width, and the internal ribs are about 4 feet apart from centre to centre.

The wrought iron through ties, which connect the ribs together so as to form a strong and compact frame, and the cross plates at the middle of the bridge, that is, at the extremity of each leaf, are shown in the plan, Plate 112. The ties have nuts and screws at the ends, and the plates are secured by wrought iron screw bolts and flanges.

The axle for the ribs is 9 inches in diameter, and cast in two pieces, joined in the middle by flanges, each 1 inch in thickness, rising 2 inches, with four bolts passing through and secured by nuts. It is supported upon solid cast iron plummer blocks, bushed with solid brass. Two enlarged views of the plummer blocks are given in Plate 111; the lower bearing pieces of the block are secured to the stone-work and counterforts by long iron bolts, with eyes and counterbolts, each 2½ inches in thickness.

The joints, trimmer, and discharging pieces of the stationary part of the bridge are each in one casting, and attached, by means of flanges, with inch and half bolts and nuts.

The bridge is raised by crab machinery at each side, acting upon a large solid quadrant wheel 4½ inches in thickness; this is secured firmly upon each end of the main axle, as well as to the front ribs of the bridge, by bolts and nuts 2 inches in thickness. The axles of the crabs are supported by bolsters and collars screwed on the outer stationary ribs of the bridge, and placed within cast iron cylindrical posts 1½ inch in thickness, which reach below the platforms of the walls of the bridge.

The upright pieces of the railing of the bridge are of cast iron, 3½ inches square, secured to the front ribs and to the platform by flanges with inch bolts and nuts; the horizontal and diagonal wrought iron bars, 1½ inch

square, are secured into the quadrant wheel and to the uprights; the intermediate bars are of inch square iron, secured in the ordinary way, and riveted into the upper and lower flat bars, which are 3 inches in breadth by ¾ inch thick, and are attached to the timber planking of the foot-way.

The ballast plates are formed of boxes of cast iron 2 inches in thickness, open at the end next the axle, supported on the main ribs by flanges, and attached by inch bolts and nuts.

The road-way is formed by timber plank 7 inches thick, secured to the upper flanges of the cast iron ribs by nuts and ¾-inch bolts.

The larger and smaller guide rails are shown in plan and section in Plate 112. They extend longitudinally on each side of the carriage-way from the commencement of the approaches at each end, and are secured to the sheeting of the road-way by bolts and nuts.

The following is a statement of the weight of iron in the several parts composing one leaf, that is, one entire half of this bridge:

	tons.	cwt.	qr.	lbs.
Weight of ribs	11	10	2	2
Balance boxes	13	13	2	4
Stationary ribs	3	5	1	12
Trimmer	1	18	0	18
Arched ribs	2	14	0	0
Wale plates	1	3	3	4
Wheel-work plates, posts, &c.	2	16	0	0
Two end frames	1	6	3	4
Centre do.	2	10	2	14
Shaft and plummer blocks, plates for do.	4	18	1	0
Guide rails, large and small	3	3	0	0
	49	0	0	2

OUSE VALLEY VIADUCT.

Plates 113, 113[a], 114, 115, and 116.

This viaduct is built over the valley of the River Ouse, on the London and Brighton Railway, at a distance of thirty-five miles from London, and about a mile N.E. of the town of Cuckfield.

The viaduct is 1437 feet in length, and is supported upon thirty-seven arches, each 30 feet span. The height from the water to the surface

of rails is about 94 feet; the parapet is 5 feet high, and the height from the surface of rails to the ground is at the north abutment 42 feet, and 38 feet at the south one.

Each pier has two courses of footings, which, taken together, are 3 feet 6 inches in depth. Where the height of the pier from the footings to the springing of the arches is greater than 48 feet, it is carried up perpendicular to this height of 48 feet below the springing.

Above this line of 48 feet below the springing, the piers are carried with a batter on each side of 1 in 48, and at each end of 1 in 24. The thickness of the upright part of the pier is 7 feet 6 inches, and above this is an offset, which diminishes the thickness to 7 feet. The length of the pier at this offset is 35 feet; its length at the springing is 31 feet, and its breadth 5 feet.

Where the height of the piers is less than 48 feet, as in the case of the two piers shown in Plate 113, the dimensions of the base are proportionably smaller than in those piers which exceed 48 feet in height. It is evident, therefore, that the length at base of any pier of less height than 48 feet will be equal to 32 feet + one-twelfth of its height, and similarly its breadth will be equal to 5 feet + one twenty-fourth of its height.

The piers are not built solid throughout, but an opening 10 feet wide is left in each pier. This opening has a semicircular arch at the top, 2 feet 3 inches thick, and a semicircular inverted one at the bottom, as shown by fig. 2, Plate 114, and the height of the opening varies according to that of the pier.

Each pier is built with a small opening or drain 4½ inches square, which descends vertically through the backing of the arches, nearly to the surface of the ground, where it is turned outwards to discharge any water which may occasionally find its way through the puddle over the arches.

The abutments have five courses of footings, the bottom course being 26 feet in breadth, and the upper one 24 feet above the footings; the abutments are carried to the springing with a thickness of 23 feet 6 inches. This great mass of brick-work is, however, lightened by four hollows, 12 feet long and 5 feet 4 inches wide, with semicircular arches at top, springing from a depth of 12 feet below the springing line of the main arches. In addition to these large openings there are two small ones at each of the angles, where the wing walls join the abutments. The plan of the small openings, as well as of the large ones, is seen in Plate 115.

The springing course for the piers and abutments is formed of moulded bricks, according to the section shown on a large scale by fig. 13, in Plate 116.

The wing walls are founded in steps, at the several depths shown in the longitudinal section, Plate 113. The plan in Plate 115 shows the thickness at the base of the several lengths of the wing walls, and fig. 6, in Plate 113, shows how these walls are carried up, with a curved batter on the face, and in vertical lines at the back, the thickness diminishing by offsets at the several levels, shown by the horizontal lines across the wing walls in the longitudinal section, Plate 113.

The pilasters are carried up perpendicular, and the curved batter on the face of the wing walls is described with a radius of about 190 feet.

The arches of the viaduct are all semicircular, with a span of 30 feet. The bricks are all moulded to the radius of 15 feet; the exterior faces, or quoins, being separately moulded to the form shown in fig. 14, Plate 116.

The arches are 2 feet 3 inches in thickness at the springing, and about half-way between the springing and the crown of the arch, an offset reduces the thickness to 18 inches.

The filling in between the arches is carried up over each pier to the height of 10 feet above the springing, and then finished off by a concave dish, with a chord of 9 feet 6 inches, and a versed sine of 1 foot.

The backing of the end arches over the abutments, to a height of 10 feet, slopes down from the top of the first arch to a height of about 15 feet above the springing line, as shown by the shaded part beneath the puddle in fig. 1, Plate 113.

At this point of 10 feet above the springing, the bounding plane of the backing suddenly changes to a more vertical direction, and meets the top of the abutment about 13 feet from the face. Four counterforts are carried up in continuation of the upper line of the backing, as far as the outside of the abutments. The plan of these counterforts is seen in Plate 115; the opening through the piers, which has been already mentioned, is continued to the top of the filling in; between the arches and the top of the opening, just in the dish, or lowest part of the curve, is fixed a grating, as shown in Plate 113.

Four interior spandril walls, 18 inches in thickness, are built upon the backing between all the arches to the height of 6 feet below the surface of the rails. These interior spandrils are built parallel with the outside walls, in such a position that one is exactly under each line of rail. These are about 3 feet 9 inches in height in the deepest part, and about

17 feet long, and the spaces between these are arched over by semicircular arches, thrown from one to the other, so as to form five hollow spaces, as seen from fig. 4, Plate 115. These small relieving arches are 9 inches thick, and the span of the two outside ones is 4 feet, that of the two next ones 3 feet 6 inches, and of the centre one 4 feet 6 inches; the crowns of these arches are on the same level as the main arches, and the space between each of these small arches is filled up solid, so as to form at the level of the tops of all the arches a level platform for the whole length and breadth of the bridge.

On the top of this levelled platform a course of puddle is laid, extending longitudinally over the whole of the arches and piers, and transversely the whole breadth between the outside spandril walls.

This puddle, which is 2 feet in thickness and 27 feet wide, is continued beyond the first arch, at each extremity of the viaduct, in a sloping direction, resting immediately upon the backing of the abutments, and upon the counterforts over the abutments, which have been already described: see fig. 1, Plate 113.

The internal spandril walls are of brick-work, 2 feet 6 inches thick, and are carried up plain to the under side of the cornice.

The cornice projects nearly 2 feet from the face of the arch, and dips about 1 inch from the horizontal, so as to allow the water to run off. Its upper part projects slightly over the face, and the under side is throated to the depth of about 1½ inch by 6 inches wide: see fig. 12, Plate 116.

The parapet is of open work, with a recess over each pier, 2 feet deep, and 3 feet wide inside. The plinth of the parapet, or that part from which the open work commences, is 1 foot 6 inches deep and the same in width.

The open work is formed of a series of small semicircular arches 1 foot wide, resting upon piers 6 inches wide by 10 inches long, and 1 foot 6 inches high.

The coping is 8 inches deep by 1 foot 6 inches wide, with a rounded top, as shown in figs. 8 and 12, Plate 116. The parapet at the back of the recesses over the piers is solid, the plinth and coping being the same in section as in the other parts of the parapet; that part between the coping and plinth is 10 inches thick, and the side walls 1 foot 6 inches wide.

Figs. 8, 9, 10, and 11, in Plate 116, will explain the form of the cantilevers for supporting the recesses. These cantilevers are of cast iron, with top and bottom plates, which are built into the brick-work of the

spandrils. These plates are bolted together, and a wrought iron tie-rod is connected transversely from one cantilever to the opposite one: see the dotted lines in fig. 8, Plate 116. The bearing part of each cantilever under the cornice of the recess has a surface of 2 feet 9 inches by 1 foot 6 inches wide.

The roofed recesses or turrets, shown in Plates 113, 114, and 115, are eight in number, one being placed over the extremity of each wing wall, and one over each abutment; their internal area is 6 feet deep by 10 feet wide; they are supported by four pilasters and three mullions, having semicircular arches between, three at the back of the turret, each 2 feet 3 inches diameter, and one at each side, 3 feet 3 inches diameter.

The pilasters are 2 feet square by 7 feet 6 inches high, surmounted by a capital, 1 foot 6 inches high; the plan of the mullion is that of a square of 1 foot, with two truncated angles: see fig. 16, Plate 116.

The form and construction of the roof for these turrets is developed in Plate 113, and in fig. 15, Plate 116.

The whole of the building, up to the height of the under side of the string course, or cornice, is composed of brick-work.

The cornice, parapet, coping, and roofed recesses, are built with Caen stone.

The railway ballasting is laid upon the course of puddle, before described, to the depth of about 1 foot 9 inches, with a small longitudinal drain tube placed between it and the puddle. The width of railway between the parapets is 28 feet 4 inches.

The rails over the viaduct are laid on longitudinal timbers, framed and scarfed together; the inclination of the railway is at this place 20 feet per mile, falling southwards.

The viaduct was built from the designs and under the superintendence of John Urpeth Rastrick, Esq., engineer-in-chief to the London and Brighton Railway; it was commenced in May, 1839, the trains for public traffic passed over it in July, 1841, and the whole was completely finished in May, 1842.

SWIVEL BRIDGE, WELLESLEY LOCK WORKS.

Plates 117 and 118.

These Plates contain detailed drawings of this bridge; it is formed of two leaves, meeting in the centre. The chord of the arc formed by the leaves is 41 feet, and its versed sine is 2 feet 3 inches; the width of the bridge between the faces of the outside ribs is 13 feet 6 inches.

The platforms upon which are laid the base rings of the bridge are formed of closely-jointed ashlar.

A plan of the roller frame is seen in Plate 117; it is 13 feet 4 inches in diameter from out to out, formed of flat bar iron 15 inches wide, with six arms meeting in the centre, also of flat iron 7 inches wide: the frame contains eight spaces for the rollers; these are of cast iron, 9 inches diameter, bored out in the eyes to receive the turned wrought iron axles, so as to move freely on them without shake. Each roller is in the form of the frustrum of a cone, 4½ inches deep, the apex of the cone being the centre of the frame.

There are five main ribs in the leaf; these are 3 feet 4 inches apart from centre to centre; each rib is cast in one piece, and of the form shown in Plate 118. The ribs are bolted down upon the traverse plates, and are connected together by means of three wrought iron ties in each leaf; these ties are 2 inches in diameter, 6 feet apart from centre to centre, and firmly secured to the outside ribs by nuts: see the cross section, Plate 118.

The back plates carry the toothed circular rack for giving motion to the bridge. The horizontal motion is derived from the vertical one by two mitre wheels. The vertical shaft is 2 inches square and 6 feet 10 inches long. The horizontal pinion is solid, with thirteen teeth, and the larger wheel with which it communicates is 5 feet in diameter, measuring from the noses of the teeth. This wheel has eight arms, and the small pinion attached to its axle is composed of fifteen teeth, which communicate directly with the circular rack before mentioned.

The girders which carry the road-way are 6 inches square, 2 feet 5 inches apart from centre to centre, and screwed down to the flanges of the ribs by bolts, countersunk at the upper side of the girders, and made fast by nuts at the under side. These are succeeded by the sheeting planks, which are 3 inches in thickness, extending in a longitudinal direction, and spiked to the girders.

The form and dimensions of the various parts composing the railing are very clearly shown in the elevation and enlarged parts of Plate 118.

Particulars of castings in one-half of swivel bridge.

	tons.	cwt.	qr.	lbs.
Weight of ribs	11	6	0	8
Upper wheel	2	10	0	0
Under do.	1	16	2	13
Two side frames	2	19	1	0
Frames for road-way	2	5	0	0
Wheels, pinions, segments, plates, posts, shafts, &c. .	1	7	1	21
End plate	1	3	2	12
Rollers	0	6	0	0
Sundries for chipping pieces, dressing, &c. . .	0	15	0	0
Small plate	1	0	1	12
	25	9	1	10

SWIVEL BRIDGE ON THE NEWRY CANAL.

BY ROBERT MALLETT, ESQ., C.E.

PLATES 119 AND 120.

This bridge is built at a skew angle of 5 degrees across the canal, behind the old market-house, at the town of Newry.

The distance between the faces of the abutments, in a line measured square to the face, is 30 feet at the level of the platform. The extreme length of each abutment is 67 feet 6 inches, and, on account of the skew, the western abutment stands 3 feet 9 inches further to the north than the eastern one, measuring in the line of the canal from a line passing through the centre of revolution of the western wing of the bridge to the corresponding centre of the eastern wing. The outside width of the road-way across the bridge is 15 feet.

The approaches at each end of the bridge have an inclination of about 1 foot perpendicular to 15 feet horizontal, and are formed of material laid in courses, rammed, rolled, and overspread with a coat of well-broken whinstone, 6 inches in thickness, which is again covered by 1 inch in depth of good binding gravel.

The masonry of the bridge consists of two principal abutments, one at each side of the canal. These are founded upon concrete, backed with

rubble, and built of hewn ashlar, the top courses of which are fitted to the swivel bridge castings.

As it was contemplated at a future period to deepen the existing bed of the canal, it was requisite to lay the foundations of the abutments at such a depth as to permit that operation to be safely performed. For this purpose the ground was cleared out to the depth of about 7 feet 6 inches below the level of the lowest part of the bottom of the canal, and upon this was laid a bed of concrete, 3 feet in thickness, composed of clean rounded pebbles, or broken limestone, sharp gravel, and pulverized hot lime, mixed in the proportion of one measure of lime, four of pebbles, and one of sharp gravel.

The road-way leading to each end of the bridge is defined by a curb-stone of good hard granite, set on edge. The stones are 6 inches in depth by 6 inches wide, and about 2 feet in length. The space between these curbings for 20 feet back from the heels of the bridge is covered by a squared stone pavement, coursed across the road-way, and bedded in fine gravel. The stone employed is good hard granite, or whinstone, each being about 4 inches thick, 6 inches deep, and 8 inches long, and all closely jointed: see Plate 120.

Each abutment is bounded in front by an exterior retaining wall or quay, the line of which is parallel with the direction of the canal. This exterior wall returns into the bank by a circular sweep of 13½ feet radius at the level of the top courses. The face of the wall has a batter of ¾ of an inch to the foot, and the first four courses diminish by offsets of 4 inches all round.

That part of the abutment which supports the base rings of the bridge consists of a retaining wall at the back, and of a space about 9 feet in width, filled with coursed rubble. The retaining wall is founded at the same depth and upon the same bed of concrete as the front wall, and the space between the two walls is built solid with coursed rubble, as shown in the section through A B, in Plate 120.

The ashlar courses in the abutment walls vary from 12 inches to 2 feet in thickness; each course is of the same thickness throughout, properly jointed, and backed up. The stones of the ashlar-work are dressed all over on the under bed, and for 18 inches on the upper bed; the vertical joints for 8 inches, and the rest scabbled, and, as well as the rubble masonry, set flush in mortar, made of well-burnt, freshly-slaked lime, free from ashes or clinkers, and clean sharp river sand, mixed in the proportion of three measures of sand to one of lime. The lime was ground in a

mortar mill, and not allowed to lie more than six days before being used.

The rubble-work is grouted with fluid mortar, made by adding water to the mortar used for the ashlar-work.

In the face of each abutment two recesses are provided for the reception of stop gates. Each of these recesses, or grooves, is 12 inches wide along the face by 10 inches deep at the level of the platform, and the back face of each is run down plumb, so that the depth increases in proportion to the batter of the wall.

The platforms to receive the base rings of the bridge consist of a course of squared closely-jointed ashlar, about 14 inches in thickness, dressed on the upper bed.

The lower beds and vertical joints are scabbled, except those of the lap stones, or coping, forming the curb, or edge of the platform over the quay walls, which latter stones are 2 feet in thickness for the breadth of the platform, and are dressed on all sides.

The breast walls at the heels of the swivel bridge, and along the land side of the inclined planes, over which the leaves of the bridge stand when open, are also of squared ashlar, in courses of from 9 to 12 inches in thickness.

The foundation of these walls is 5 feet below the level of the platforms. The curve of the breast walls is accurately formed so as to suit the sweep made by the heel of the swivel bridge, and the quoins, where they meet the face of the quay walls, are rounded off to a radius of 2 feet on the plan.

The coping all round the circular and straight retaining walls of each abutment, as well as that of the circular breast walls, consists of a course of thorough stones, dressed all round, about 16 inches in thickness and 3 feet 6 inches wide, with their front arris rounded to a radius of 3 inches.

Recesses for the gearing to move the swivel bridge are formed in each breast wall, as shown in the plan on Plate 120.

The whole surface of each abutment within the coping is laid with large hewn, close-jointed pavement of well squared blocks of stone, about 6 inches wide by 9 inches long. The surface of this pavement has an inclination outwards of 1 foot in 40. The coping of the breast walls forms the termination of this pavement on the land side: the dimensions of this coping, which extends round the recesses for the moving gear, have been before stated.

The fender posts fixed in this coping, and corresponding in position

with the pedestals of cast iron, containing the moving gear in each abutment, are of the best and soundest granite; an elevation of one of these is shown in the front elevation of western abutment in Plate 120.

The superstructure of the bridge consists of two nearly similar leaves, or parts; the span of the arch formed by these when closed is 31 feet 4 inches on the chord, and its versed sine is 4 feet $1\frac{1}{2}$ inch, the radius of the circle being 32 feet 4 inches.

The base ring of each abutment is 11 inches in width all round, 11 feet 2 inches outside diameter, with a raised rib round its upper surface, at the inner edge. This ring is sunk into the top course of masonry, so that its upper surface is flush with the surface of the stone-work, as shown in the transverse section through A B, in Plate 119.

Upon this ring rests another one, called the abutment ring. This latter is a hollow casting, with vertical ribs at every 12 inches, connecting the upper and lower flanges. The abutment ring is bedded upon the bed ring by means of twenty-four flat wrought iron keys, filed and fitted to their places, and resting upon properly filed key beds on the bed and abutment rings, so as to enable the latter to be adjusted as to level at any time: twenty-four other short keys are fitted and driven vertically between the projecting vertical rib of the bed ring and the inner edge of the base ring, so as to adjust the position of the abutment rings, and finally to secure them in their proper positions in the horizontal direction.

The abutment ring, as well as all the other rings, is cast in one piece: see general plan of framing, Plate 119.

The upper and lower rings are similar, each being a ring of cast iron of 10 feet 10 inches diameter from out to out, and in section a rectangle of 5 inches wide by $2\frac{1}{2}$ inches deep, turned smooth and true in a lathe on the sides bearing against the rollers, and bolted at intervals of 2 feet to the abutment ring and to the upper traverse ring respectively, by $\frac{3}{4}$-inch countersunk bolts, the heads of which are filed off fair with the face. Each ring is fairly and uniformly bedded upon the abutment and traverse rings on the seats prepared to receive them. Each leaf rests upon twenty-four cast iron rollers of 8 inches diameter, bored out in the eyes to receive turned wrought iron axles of $1\frac{1}{8}$ inch diameter. Each roller is provided with a raised lip or flange at the inner edge, formed to the proper curve.

The roller frame consists of an inner and an outer ring of flat bar iron, having a space between them when placed concentric for the rollers, together with a loose washer of $\frac{1}{4}$ of an inch thick at each side; the outer ring is 3 inches by $\frac{7}{8}$ inch, and the inner one 3 inches by $\frac{3}{4}$ inch.

The axle pins of the rollers pass through and connect both these rings, and are placed strictly in radial lines to their circumferences, passing through holes elongated in their vertical diameters by $\frac{1}{10}$ of an inch, so as to allow a little up and down play for the roller pins, if required. Each roller pin is case-hardened, having a sound head at the inner end, and a corresponding washer, and pin of wrought iron passing through both washer and pin, and riveted over, so as to secure the pins into and through the roller frame. Each roller is provided with an oil hole, bored between two of the four arms into the eye.

The upper traverse ring, which carries the upper roller ring upon its under surface, is shown in the general plan of framing, Plate 119, and is provided with all the requisite brackets, jaws, &c., to receive and secure the main ribs, and also with one main cross rib, passing along its diameter, and having a broad eye in its centre to receive the main centre pin, round which the leaf revolves.

Concentric with the roller rings, and sunk $1\frac{1}{2}$ inch into the top course of the masonry of the platform, is placed a cast iron cross of four arms, carrying the main centre pin of wrought iron, $4\frac{1}{2}$ inches in diameter.

The pin is fitted to it by boring and turning, and keyed into the cross, which is bored in a slightly tapering form to receive it, and was heated before the pin was driven in and keyed. The cross is effectually secured down to the upper courses of the masonry of the platform by four $1\frac{1}{4}$-inch round bolts of wrought iron, leaded into jumped holes, and provided with large cast iron washers below the stones of the course, and carrying hexagonal nuts above.

The centre pin projects up through the traverse ring, and secures it down by means of a large turned and bored cast iron washer, cottered by a double cotter through the pin. A space or freedom of $\frac{3}{8}$ inch is left between the upper surface of the traverse frame and the under side of the washer; the eye of the traverse frame, as already stated, being bored to receive the centre pin, and leave a space of $\frac{1}{8}$ of an inch all round.

The bridge consists of four main ribs in each leaf; the two inner ones are 5 feet apart from centre to centre, and from these to each of the outer ones the distance is 2 feet 6 inches from centre to centre. The ribs are shown in section in the transverse section through C D, Plate 119, and the elevation of one of the exterior ribs is also given in the same Plate.

Each rib is cast in one piece, with the requisite lugs, jaws, flanges, bolt holes, &c. The ribs are bolted down upon the traverse ring with $\frac{7}{8}$-inch iron bolts, and all the joints are well fitted and bedded.

The two inner ribs are connected together by means of a diagonal frame, through the upper and lower parts of which are passed cross tie-bolts of 1⅛ inch diameter, of wrought iron, which bolts also pass through distance pipes, connecting the inner with the outer ribs: see transverse section through C D, and general plan of framing, Plate 119. The ribs are further strengthened and secured by five sets of distance pipes; these are also shown in the Plate last referred to, and are attached to the ribs by means of flanges; and cross tie-bolts pass through them, of the same dimensions as those described above.

The lower parts of all the four ribs at the springing of the arch are secured together by the abutment plate, which is cast truly to the curve of revolution described by the radius of the arch at the abutting front where the leaf revolves. It is bedded firmly and solidly upon the abutment pieces or blocks attached to the abutment ring, as before described, when the bridge is closed. The leaf when closed abuts against five of these blocks, and when fully open it abuts against two only.

The abutment plate is cast in one piece, and is provided with suitable flanges to receive and secure it to those of the ribs by 1⅛-inch bolts.

The outer ribs are provided with dovetailed recesses at distances of about 4 feet, into which are fitted, by chipping and filing, cast iron brackets, which carry the outer ends of the road-way planking. The outer extremities of the brackets are connected together by fascia plates (see transverse sections through A B and C D), having a rabbate at the upper side to receive and defend the ends of the planking, and neatly moulded at the lower edges. The uprights of the railing pass through both the brackets and the horizontal flanges of the fascia plates, so as to secure them together by nuts and screws, into which the extremities of the uprights are formed.

An elevation of the meeting plate is given in Plate 119. It is cast in two parts, each with butt joints in the middle; one half of the meeting plate is straight, and at right angles to the centre line of the bridge, and the other half is swept to a circle of 22 feet 6 inches radius, the centre being that upon which the bridge turns; the curved part of the meeting plate for the western abutment is convex, the corresponding part of the eastern one being concave. A freedom of ⅛ of an inch is left between both meeting plates when closed, except at seven equidistant points, where chipping strips of 1½ inch wide are filed and bedded so as to abut firmly.

The back plate is represented in Plate 119; it is cast in two pieces, with a butt joint in the middle; it is accurately swept to a radius of 10 feet 6 inches, the centre being the point upon which the bridge revolves. The back plate is secured to and connects the back ends of the main ribs by 1-inch bolts and nuts, passing through the flanges formed upon the latter to receive them. The outer ends of the back plates are cast separately, and carry the extremities of the fascia plates and of the road-way planking, being further supported and secured by two diagonal stays abutting against the outer ribs. The lower edge of the back plates, as well as the adjacent parts of the main ribs at their under edges, are furnished with a flange all round, to support perforated plates of cast iron $\frac{7}{8}$ of an inch thick, and bolted to them with $\frac{3}{4}$-inch bolts and nuts, and intended to carry the ballast required for balancing the bridge.

The ballast consists of heavy blocks of squared and chisseled granite stone.

At their upper edges, below the level of the fascia plates, the back plates carry the large toothed circular racks for giving motion to the bridge; the breadth of the rack is $3\frac{1}{2}$ inches and the pitch 2 inches: the curve is formed to a radius of 10 feet 6 inches, measuring to the noses of the teeth, the centre being the centre of motion, and the length of the chord is 17 feet. The rack is cast in four lengths, and fitted, by chipping and filing, upon proper strips to the back plate, and secured by $\frac{3}{4}$-inch countersunk bolts and nuts.

The road-way consists of two thicknesses of planking, the lower one being a layer of $2\frac{1}{4}$ inches in thickness of British oak, laid diagonally, and secured by $\frac{3}{8}$-inch countersunk bolts and nuts to the upper flanges of the main ribs, fascia plates, back plates, and meeting plates. Each plank is in one length across, and about 9 inches wide, planed all over, shot straight on the edges, and free from all defects.

The joints were all caulked with oakum, in the usual way, when the planks were bolted down.

The cast iron wheel strike, or curb, which separates the foot-paths from the carriage-way, is bolted down along the edges of the carriage-way upon the oak planking by $\frac{3}{4}$-inch bolts and nuts with countersunk heads above, at 2 feet distance. The curb is cast in lengths of about 8 feet, and is shown in section in the transverse sections, Plate 119. They are placed in parallel lines, 10 feet apart, measured across the road-way.

The whole space between the curbs, for the length and breadth of the bridge, is covered over with a layer of thick brown paper, saturated in

boiled coal tar, which was laid upon the oak plank whilst the coal tar was warm; the paper was spread smoothly, and overlapped about 1 inch at the joints, the oak plank having previously received a coat of coal tar at boiling temperature. The second layer of planking was laid over this, and spiked down to the oak planking; this planking is of sound elm, free from sap or defects, in planks about 9 inches wide by $2\frac{1}{4}$ inches thick, laid at right angles to the line of the bridge, planed all over, shot straight upon the edges, and well fitted at the ends to the cast iron curbs. It is spiked down to the oak planks with rose-headed spikes of 5 inches extreme length, driven in alternately at 18 inches apart, within 2 inches of the edge of the curb plank, into holes properly bored to receive them.

The nosing plate is of wrought iron 3 inches wide by $\frac{3}{8}$ inch thick, and is let flush into the plank across the whole breadth of the bridge, over the back plates and meeting plates, to protect the arris of the planking, and is secured down by $\frac{3}{8}$-inch countersunk bolts and nuts. A corresponding plate of cast iron, 15 inches wide, is let in flush upon the top course of stone of each breast wall at the heel of the bridge, to protect the arris of the masonry. It is swept to the proper curve, and roughed upon the upper side to prevent the slipping of animals. It is leaded into the stone, and further secured by ten rag bolts, with sunk nuts, also leaded into the stone in holes jumped to receive them.

One extremity of the locking plate is hinged, and formed with a copper padlock in each wing; the padlock is secured to the staple by 12 inches of chain, and is provided with duplicate keys, so as to prevent the bridge from being opened at any time without authority.

The railing is of wrought iron; the upright bars, secured as already described, are of flat iron, alternately 2 inches by $1\frac{1}{8}$ inch and 2 inches by $\frac{3}{4}$ inch; through these uprights pass three bars of $\frac{3}{4}$-inch round iron, running parallel the whole length in one piece, and having a screw with a projecting thread at the intersection with each upright, by which they are firmly secured to the same by two nuts. A pair of diagonal bars of flat iron, 2 inches by $\frac{3}{4}$ inch, is also fixed at each end of the railing, and through these the horizontal bars are also passed, and the diagonals are secured at their extremities to the uprights by nuts, in the manner described for the horizontal bars.

The whole line of railing is fair, straight, and upright.

The revolving gear consists of a pinion of 10 inches diameter and 2 inches pitch, working into the toothed rack upon the back plate; at the bottom of the same vertical shaft of wrought iron, 3 inches diameter at the

smallest part, is a spur wheel of 5 feet diameter, $\frac{3}{4}$ inch wide on the tooth, and $1\frac{3}{4}$ inch pitch; the latter works a pinion of 8 inches diameter fixed at the bottom of a vertical shaft of wrought iron $2\frac{3}{8}$ inches in diameter at the smallest part, and carrying at its upper end a crown or bevel wheel of 18 inches in diameter, which again works into another bevel pinion of $4\frac{1}{2}$ inches diameter, placed upon a short horizontal shaft, one end of which is provided with a square to receive a moveable wrought iron winch handle, the radius of which is 16 inches. This arrangement is shown very clearly in an enlarged view, in Plate 119.

The gear pedestals are cast hollow, of a cylindrical form. The cap, which is dish-shaped, is cast separate, so as to lift off, and is secured by four lugs inside and $\frac{3}{8}$-inch countersunk bolts. The pedestal stands upon a bed plate of cast iron, and the several shafts are sustained in bearings provided with brasses; they are turned true, and secured by bolts and nuts.

All the wheels and pinions have bored eyes, and are keyed upon turned shoulders, with proper projections upon the shafts. The lower ends of the vertical shafts rest in hard brass foot-steps, let into a cast iron foot-box, sunk, and leaded into the stone-work. The bed plate of the pedestal is sunk flush into the top courses of the breast walls, and leaded, and further secured by seven $\frac{7}{8}$-inch rag bolts with sunk nuts, leaded into holes jumped to receive them.

Two cast iron spuds or stops are provided, one for each wing, and leaded into the stone of the platform, and further secured with three 1-inch rag bolts, leaded into the stone and placed to receive the under part of the outer rib near the abutment plate when the bridge is opened, to prevent its being overturned.

The whole of the upper surface of the road-way, when completed, was payed over with two coats of boiling-hot coal tar, which had been for some hours boiled, with finely-pulverized quicklime, in the proportion of one pound of lime to a gallon of tar; and as soon as this coat was laid on, it was sifted over with fine dry sharp sand, over which, when dry and the superfluous sand had been removed, the second coat was laid on in a similar manner.

The under side of the road-way plank was painted with three coats of coal tar, boiled as above described, and having the addition of four ounces of red lead to each gallon of tar.

The whole of the iron-work of the bridge, in every part, received three coats of good oil paint, the last coat being of a light lead colour.

All the castings of the bridge are of bright gray iron, sound and free from defect, cast straight, fair, and out of winding.

The wrought iron is of the best English bar iron, or scarfed iron, and the forging free from defects, and the nuts in every part of the bridge are hexagonal.

The cost of this bridge was £2238.

ONE OF THE WIRE BRIDGES OVER THE FOSSE AT GENEVA.

Plate 121.

This is a suspension bridge with two equal openings, each of 132½ feet, erected by Colonel Dufour, a French engineer. The platform of the bridge is about 300 feet in length from end to end, and being only intended for foot passengers, is no more than 7½ feet in width.

The main suspension cables of this bridge are four in number, namely, two on each side of the platform. They are placed one over the other, about 12 inches apart, and are composed of wires $\frac{1}{12}$ of an inch in diameter, laid parallel and close together, and bound round by spiral wire. The upper cables are 1 inch in diameter, and the lower 1 inch and $\frac{1}{8}$. The position of the upper and lower cables, and the form and dimensions of one of the radiating suspenders connecting the two cables together, will be clearly seen from one of the figures in the Plate, where these details are shown on a scale of one-third the full size. Each of the abutments, as well as the pier in the centre, is built of masonry, with an arch over the platform of the bridge. It will be observed from the elevation that at one end of the bridge, where the toll-house is erected, the abutment is much more massive than at the other end. The toll-house is at that end of the bridge which adjoins the town, and the abutment here is made of much larger size than the other, on account of the method in which the cables are terminated by upright back stays instead of sloping stays, as at the other end. The upright back stays are fixed close against the back of the abutment. They are four in number, namely, one to each cable, and are made of wrought iron 1$\frac{3}{8}$ inch square. Their lower extremities are carried about 5 feet into the ground and strongly secured to horizontal iron beams, laid edgeways in the foundation of the abutment. One of the small figures in the Plate shows the connexion of a cable with one of the upright back stays. As the abutment at this end of the bridge has to withstand the whole drag and weight which at the other end is partly

transferred to the mooring masonry of the back stays, the reason for making it much stronger and more massive than the other abutment will appear sufficiently evident. The abutment at the toll-house end of the bridge not only has to support the weight of the chains passing over its arch, but is itself the mooring anchor which has to resist the whole drag due to the weight and deflection of the cables. At the other end of the bridge, the cables, after passing through the masonry over the arch of the abutment, take a sloping direction for about 5 feet, and are then united by coupling links to the iron back stays, which are each 1¼ inch square, and are continued to the surface of the ground and several feet below it, in the same sloping direction. The form of the coupling links and of the loops in the cables where they are attached to the sloping back stays, is shown by one of the figures in the Plate. The other coupling links, shown in the Plate where two parts of the cable were united, are those which were used over the middle pier.

The vertical suspending rods which connect the platform of the bridge with the lower cables are of ¾-inch round iron; they are attached to the lower cable at the extremity of each radiating suspender between the two stirrups of the latter, as shown in the Plate. A circular-headed strap encircles the lower cable at the junction of each suspending rod, and is fixed to the latter by a nut and screw bolt, as shown by the enlarged section in the Plate. The lower extremities of the suspending rods pass through the transverse bearers which support the platform, and are secured below by a nut which is screwed on at the under side of an iron plate. Longitudinal joists rest upon these transverse bearers, and are covered by transverse close planking which forms the surface of the platform. The standards for the iron railing are bolted through the outside joists, through the transverse bearers, and also through a side rail below the latter. This side rail is 6½ inches deep by 4¼ wide, and serves to give additional stiffness to the framing of the platform. The details of the outside timber-work of the platform and of the iron railing are clearly shown by the sketches in the Plate. Below the platform is a series of diagonal ties of wrought iron ⅞ of an inch in diameter, arranged in a polygonal form, and connecting the platform of the bridge with a lower part of the pier and abutments, as seen in the elevation.

This bridge, in common with the other erected by Colonel Dufour at Geneva, differs in several important particulars from the wire bridge at Fribourg, which is the work of M. Chaley, also a French engineer. In the Geneva bridges, the suspension rods are round bars of iron instead of

being mere cords of wire, as at Fribourg. The suspension cables are also differently disposed, being suspended at Geneva one over the other instead of being united into one main cable, as in the Fribourg bridge. It is generally understood, however, that the French engineers, who have had more experience than any others in the construction of wire bridges, do not approve the practice of placing the cables over each other. The principal objection alleged against this practice is the unequal expansion of the two cables when the sun is in the plane of the two, that is, in such a position as to be shining full upon one while the other is completely in shade. Under these circumstances, it is said that the upper cable is dilated much more than the other, and that this occasions a lowering and slackening of the upper cable, which throws all the weight of the bridge upon the lower one.

In fabricating the cables of his bridges at Geneva, Colonel Dufour was extremely particular in arranging the wires with perfect parallelism, so that no jamming or twisting of the wires together should afterwards take place. He observes, in his Memoir on the Suspension Bridges at Geneva, that he had never seen in France any wire cables at all comparable to those which he himself employed. The threads, he says, are commonly ill-arranged, and are consequently subject to great inequality of tension, and numerous vacuities are left through which water penetrates into the interior of the cable. In order to avoid these inconveniences, Colonel Dufour employed sheets or frames of metal, perforated with a number of small openings through which the wires were passed. A frame of this kind is fixed at each extremity of the cable, and a third one is moveable backwards and forwards into any required position, so as to keep the whole of the wires separate and parallel up to the time when they are to be bound together with a coil of spiral wire.

Some of the French engineers disapprove of the method of disconnected ligatures which have been adopted for binding the threads of the cable together in both the suspension bridges described in this work. It has been observed, when the ligatures are several feet apart, as in the Fribourg bridge, and even where the ligature is a spiral envelope with coils not more than an inch apart, as in the cables of the Geneva bridges, that the wires, being acted upon by variations of temperature and other causes, do not continue in such a perfectly close and parallel condition as to prevent moisture from penetrating between them. To remedy this defect, M. Vicat has proposed to envelope the cables with thin metallic tubes; and some

other engineers are of opinion that a close and continuous spiral envelope would be preferable to any other contrivance. M. Boudsot, civil engineer, the author of an able article on Suspension Bridges in the Revue Générale de l'Architecture, mentions a wire cable upwards of 260 feet in length, which is entirely enveloped by a close continuous spiral covering, so perfectly executed as entirely to prevent the penetration of water. This cable is employed for the ferry-boat at Chatillon sur la Loue, and has the appearance of a long cylindrical rod with the flexibility of a hempen rope.

WESTERN RAILROAD CONNECTICUT RIVER BRIDGE.

Plate 122.

In a former part of this work (Plates 2, 3, and 4), will be found drawings of Mr. Town's lattice bridge, and the design now before us is intended to show the construction of another kind of timber-frame bridge which has been extensively adopted in America, and is known under the name of Long's Frame Bridge.

It will be observed that this design differs from Mr. Town's principle in having a smaller number of diagonal braces, their place being supplied by vertical wrought iron ties.

The bridge, from which is taken this example of Long's framing, consists of seven equal openings of 180 feet, measuring from centre to centre of the piers. Piling is driven under each of the piers, and their foundations are protected by mounds of rubble thrown in round the footings.

The bottom string course in this bridge is a beam built of six planks, each a foot in depth, four of them being 5 inches in breadth, and the two outside planks each 4 inches. When these planks are put together they form a beam 12 inches in depth by 28 in breadth. The planks are bolted together by screw bolts, placed about 2 feet apart, and alternately near the top and bottom of the beam, as shown in the vertical section. Short transverse blocks of wood, of a triangular shape, are let into this bottom string to the depth of about 1 inch, and into the sides of these blocks are mortised the ends of the diagonal braces, as also shown in the vertical section. These diagonal braces abut against similar blocks which are let into the top string beam. The top string consists of three lines of 8-inch square timber placed with a small space between each, so as to make the whole breadth of the beam 28 inches, the same as that of

the lower string. The pieces composing this beam are bolted together at intervals of 7 feet.

All the braces are 8 inches square. The number of braces abutting on each block both at the top and bottom string course is always three, namely, two on one side and one on the other, as shown on the plan of bottom string. Along the top string beam are fixed short cross pieces 5 inches by 7 inches, one above each of the abutting blocks before mentioned. These cross pieces receive the tops of the vertical ties, which pass entirely through the framing from top to bottom, and are secured above the top string and below the bottom one by screw bolts and nuts. Similar cross pieces below the lower string receive the extremities of the vertical ties. Two of these vertical ties pass through each of the abutting blocks, so that for the two sides of the bridge there are four vertical ties in each length of 7 feet.

The frames are connected at the top by cross beams, and the lateral stiffness of the bridge is further increased by wrought iron ties at each of the piers, carried into the masonry at some depth below the platform of the road-way. The longitudinal sleepers for the railway, which has only a single track over this bridge, are laid upon transverse beams which rest upon the bottom string pieces, and are placed one on each side of each of the abutting blocks throughout the whole length of the bridge. The platform is further strengthened by diagonal horizontal beams framed between the bottom strings, as shown in the general plan.

The kind of frame bridge here described possesses, in common with Town's lattice bridge, the advantage of great stiffness and strength at an expense of a comparatively small quantity of materials. Neither bridge exerts any lateral thrust against the piers or abutments, and each presents about equal facilities for repairing and restoring, without injury to the rest of the structure, any separate part which may ever be decayed or injured.

Mr. David Stevenson states[5] that the white pine (*pinus strobus*), which grows in great abundance and perfection in the United States, is generally considered best suited for the construction of frame bridges. The preference is given to this timber on account of its lightness and rigidity, and also because it is found less liable to warp or cast on exposure to the atmosphere than most other timber of that country.

[5] Notice relative to Long's American Frame Bridge.—Edinburgh New Philos. Journal, April, 1841.

In the construction of frame bridges on this principle, it is necessary to provide a temporary work of piling entirely across the river or ravine over which the bridge is to be placed. This is to support the frames during their erection. In the mean time, the several frames for each opening must be accurately fitted and put together in the carpenter's yard. When the piers and abutments have been carried up to the proper height to receive the platform, the frames are then taken to pieces and re-erected in their permanent positions.

GENERAL INDEX.

The following recapitulation of the Contents of these volumes, with the number of pages comprised in the separate portions of the work, will exhibit to the reader the plan of the General Index.

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VOL. II.

SUPPLEMENT.

GENERAL INDEX.

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